PROCEEDINGS

OF THE

AMERICAN SOCIETY

OF

CIVIL ENGINEERS

VOL. XXXVIII—No. 10

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December, 1912

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DECEMBER, 1912

Edited by the Secretary, under the direction of the Committee on Publications.

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NEW YORK 1912

American Society of Civil Engineers

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The House of the Society is open from 9 A. M. to 10 P. M. every day, except Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.

HOUSE OF THE SOCIETY—220 WEST FIFTY-SEVENTH STREET, NEW YORK.

TELEPHONE NUMBER....................5913 Columbus.

CABLE ADDRESS....................."Ceas, New York."

*Filling the unexpired term of Vice-President Alfred P. Boller, who died December 9th, 1912.

†Vacancy caused by the death of Vice-President A. P. Boller.

‡Vacancy caused by the death of Director George A. Kimball on December 3d, 1912.

Vol. XXXVIII. DECEMBER, 1912. No. 10.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PROCEEDINGS

This Society is not responsible for any statement made or opinion expressed in its publications.

SOCIETY AFFAIRS

CONTENTS

MINUTES OF MEETINGS

OF THE SOCIETY

November 20th, 1912.—The meeting was called to order at 8.30 P. M.; A. L. Bowman, M. Am. Soc. C. E., in the chair; Chas. Warren Hunt, Secretary; and present, also, 153 members and 21 guests.

A paper by A. W. Buel, M. Am. Soc. C. E., entitled "The Sewickley Cantilever Bridge Over the Ohio River," was presented by the Secretary, and illustrated with lantern slides. The paper was discussed orally by Messrs. C. W. Hudson and Lewis D. Rights, and written communications on the subject from Messrs. L. J. Le Conte, Charles Worthington, and Theodore A. Straub, were read by the Secretary.

A paper entitled "Ports of the Pacific," by H. M. Chittenden, M. Am. Soc. C. E., assisted by A. O. Powell, M. Am. Soc. C. E., was presented by title by the Secretary, who also read written communications on the subject from Messrs. L. J. Le Conte and Lewis M. Haupt. The paper was discussed orally by E. P. Goodrich, M. Am. Soc. C. E., who illustrated his remarks with lantern slides.

Adjourned.

December 4th, 1912.—The meeting was called to order at 8.30 P. M.; Director T. Kennard Thomson in the chair; Chas. Warren Hunt, Secretary; and present, also, 96 members and 16 guests.

The minutes of the meetings of October 16th and November 6th, 1912, were approved as printed in *Proceedings* for November, 1912.

A paper entitled "Tufa Cement, as Manufactured and Used on the Los Angeles Aqueduct," by J. B. Lippincott, M. Am. Soc. C. E., was presented by title. The Secretary read communications on the subject from Messrs. L. J. Le Conte and J. M. O'Hara, and the paper was discussed orally by Messrs. Luther Wagoner, E. D. Knap, G. A. Flynn, W. J. Boucher, and O. E. Mogensen.

A paper by E. G. Hopson, M. Am. Soc. C. E., entitled "The Economic Aspect of Seepage and Other Losses in Irrigation Systems," was also presented by title. Communications on the subject from Messrs. L. J. Le Conte and W. C. Hammatt, were read by the Secretary, and the paper was discussed orally by Luther Wagoner, M. Am. Soc. C. E.

The Secretary announced the election of the following candidates on December 3d, 1912.

As Members

Daniel Wheeler Bowman, Phœnixville, Pa.
William Stone Ide, Detroit, Mich.
Joseph Harris Kimball, Louisville, Ky.
Harry Snedden Laird, Cheat Haven, Pa.
John Mead, Dallas, Tex.
Lorenzo Perez Castro, City of Mexico, Mexico
Henry Gottlieb Raff, New York City
Carl Jonas Rhodin, San Francisco, Cal.
Walter Scott Wheeler, Lima, Peru
Francis Charles Williams, Sheridan, Wyo.
Frank Gordon Wolfe, Scranton, Pa.

As Associate Members

Normand Daggett Brainard, Springfield, Ohio
Elmer Hovey Brown, Hempstead, N. Y.
Fredric Sidney Burroughs, Olympia, Wash.
James Francis Curley, Pittsburgh, Pa.
Charles Stephen Dandois, Salladasburg, Pa.
Barry Dibble, Minidoka, Idaho
Robert Moore Dunham, Fort Worth, Tex.
William Howard Durbin, Evansville, Ind.
Francis Seeley Foote, Jr., Berkeley, Cal.
William Bradley Freeman, Bangkok, Siam
Fred Allen Gorham, Pompeys Pillar, Mont.
Ambrose Goulet Grandpré, Chicago, Ill.

Harry Rutledge Hall, Baltimore, Md.
Ivan Guy Harmon, Denver, Colo.
Ralph Zenas Kirkpatrick, Paraiso, Canal Zone, Panama
Geoffrey Wainman Mayo, Manila, Philippine Islands
Irwin Selden Osborn, Columbus, Ohio
Charles Joseph Renner, New Rochelle, N. Y.
Ned Hensel Sayford, York, Pa.
Benjamin Franklin Schaberg, St. Louis, Mo.
Brahma Nand Sharma, London, W. C., England
Milton Frederick Stein, Pittsburgh, Pa.
Kimbrough Enoch Voorhes, McGill, Nev.
Thomas Isaac Weston, Columbia, S. C.

As Juniors

Fritz Muss Arnolt, New York City
Arthur Francis de Jongh, Banes, Cuba
George Joseph Fisher, Coyote, Cal.
John Raymond James, Detroit, Mich.
Paul Sidney Jones, Fort Collins, Colo.
Harootun Hovhannes Khachadoorian, Burlington, Vt.
Lester William Perrin, Toronto, Ont., Canada
Karl Lewis Ponzer, Brinkley, Ark.
Daniel Henry Seaman, Newark, N. J.
Kirby Baldwin Sleppy, Los Angeles, Cal.
Henry Lawrence Thackwell, Chelan, Wash.
Franklin Thomas, Birmingham, Ala.
Newton Benjamin Wade, Millville, N. J.
John Croswell Warkley, Cheyenne, Wyo.
Alexander Woodward Yereance, Clanton, Ala.

The Secretary announced the transfer of the following candidates on December 3d, 1912:

From Associate Member to Member

Weston Earle Fuller, New York City
Oliver Thomas Reedy, Maxville, Mont.
Edward Augustus Southworth, Hilo, Hawaii

From Junior to Associate Member

Carl August Bock, St. Marc, Haiti
John Worde Caldwell, Honolulu, Hawaii
Rafael Sanchez Giquel, Havana, Cuba
Julius Reed Hall, Chicago, Ill.
Thomas Leach, Montreal, Que., Canada
Norris Raymond Macklem, Manila, Philippine Islands
William Watters Pagon, Baltimore, Md.

Earl Patterson, Selden, N. Mex.
Miguel Villa, Havana, Cuba
Albert Jones Willis, New York City

The Secretary announced the following deaths:

Daniel Seymour Brinsmade, elected Member, February 1st, 1888; died September 7th, 1912.

George Albert Kimball (*Director*), elected Junior, May 12th, 1875; Member, July 1st, 1891; died December 3d, 1912.

Adjourned.

December 18th, 1912.—Because of the necessity of going to press with this number of *Proceedings* in advance of this meeting, the publication of its minutes must be deferred until January, 1913. Two papers have been set down for discussion: "Prevention of Mosquito Breeding," by Spencer Miller, M. Am. Soc. C. E.; and "The Sanitation of Construction Camps," by Harold Farnsworth Gray, Jun. Am. Soc. C. E.

OF THE BOARD OF DIRECTION

(Abstract)

December 3d, 1912.—Vice-President Churchill in the chair; Chas. Warren Hunt, Secretary; and present, also, Messrs. Bush, Endicott, Gerber, Knap, Loomis, and Snow.

It was decided that the next meeting of the Board be held on the evening of January 7th, 1913, instead of December 31st, 1912.

It was decided that the first meeting of the Society in January be held on the evening of Wednesday, January 8th, instead of on January 1st, 1913.

President John A. Ockerson was nominated as a member of the John Fritz Medal Board of Award to take the place of Past-President Charles Macdonald, whose term of office on that Board will expire January 17th, 1913.

The following resolution which was unanimously adopted by the meeting of the Society, November 6th, 1912, was considered:

"*Resolved,* That a Special Committee of seven be appointed to codify present practice on the bearing value of soils for foundations, and report upon the physical characteristics of soils in their relation to engineering structures."

It was moved, seconded, and carried, that it is the sense of the Board that this Committee be appointed.

The Secretary reported that on November 8th, 1912, he had forwarded to Dr. J. H. T. Tudsbery, Honorary Treasurer, $900, being the result of 160 subscriptions from members of this Society to the proposed Lord Kelvin Memorial to be erected in the form of a memorial window in Westminster Abbey.

Messrs. George W. Tillson, Arthur S. Tuttle, and Chas. Warren Hunt, were appointed a Committee to take charge of the arrangements for the next Annual Meeting.

The following resolution adopted at the Annual Meeting of 1912 was considered:

"*Resolved,* That the Board of Direction be asked to consider a recurrence of the practice of providing a luncheon on the first day of the Annual Meeting."

On motion, duly seconded, it was resolved that it is the sense of this Board that the providing of this luncheon is not advisable.

The resignations of 4 members, 2 Associate Members, and 1 Associate, were accepted.

Ballots for membership were canvassed, resulting in the election of 11 Members, 24 Associate Members, and 15 Juniors, and the transfer of 10 Juniors to the grade of Associate Member.

Three Associate Members were transferred to the grade of Member.

Applications were considered, and other routine business transacted.

Adjourned.

ANNOUNCEMENTS

The House of the Society is open from 9 A. M. to 10 P. M., every day, except Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.

FUTURE MEETINGS

January 8th, 1913.—8.30 P. M.—A regular business meeting will be held, and a paper by H. T. Cory, M. Am. Soc. C. E., entitled "Irrigation and River Control in the Colorado River Delta," will be presented for discussion.

This paper was printed in *Proceedings* for November, 1912.

Wednesday and Thursday, January 15th and 16th, 1913.—The Sixtieth Annual Meeting will be held. The Business Meeting will be called to order at 10 o'clock on Wednesday morning at the Society House. The Annual Reports will be presented, officers for the ensuing year elected, members of the Nominating Committee appointed, Reports of Special Committees presented for discussion, and other business transacted.

Arrangements for the Annual Meeting have been placed in the hands of the following committee: Messrs. George W. Tillson, Arthur S. Tuttle, and Charles Warren Hunt.

February 5th, 1913.—8.30 P. M.—This will be a regular business meeting. Two papers will be presented for discussion, as follows: "Characteristics of Cup and Screw Current Meters; Performance of These Meters in Tail-Races and Large Mountain Streams; Statistical Synthesis of Discharge Curves," by B. F. Groat, Assoc. M. Am. Soc. C. E.; and "The Infiltration of Ground-Water into Sewers," by John N. Brooks, Jun. Am. Soc. C. E.

These papers are printed in this number of *Proceedings*.

February 19th, 1913.—8.30 P. M.—Two papers will be presented for discussion at this meeting, as follows: "A Suggested Improvement in Building Water-Bound Macadam Roads," by J. L. Meem, Esq.; and "On Long-Time Tests of Portland Cement," by I. Hiroi, M. Am. Soc. C. E.

These papers are printed in this number of *Proceedings*.

SPECIAL MEETINGS FOR TOPICAL DISCUSSION

On the two days immediately following the Annual Meeting, three meetings of the Society will be held, at which the subject for discussion will be "Road Construction and Maintenance."

The meetings will be held as follows:

First Meeting, Friday, January 17th, 1913.—10 A. M.—The following sub-division of the subject will be discussed:

(1) "Cement-Concrete Pavements."

Second Meeting, Friday, January 17th, 1913.—2 P. M.—Two sub-divisions of the subject will be discussed:

(2) "Cost Records and Reports."

(3) "Design of Highway Systems."

Third Meeting, Saturday, January 18th, 1913.—10 A. M.—The following sub-division of the subject will be discussed:

(4) "Equipment for the Construction of Bituminous Surfaces and Bituminous Pavements."

SEARCHES IN THE LIBRARY

In January, 1902, the Secretary was authorized to make searches in the Library, upon request, and to charge therefor the actual cost to the Society for the extra work required. Since that time many searches have been made, and bibliographies and other information on special subjects furnished.

The resulting satisfaction, to the members who have made use of the resources of the Society in this manner, has been expressed frequently, and leaves little doubt that, if it were generally known to the membership that such work would be undertaken, many would avail themselves of it.

The cost is trifling compared with the value of the time of an engineer who looks up such matters himself, and the work can be performed quite as well, and much more quickly, by persons familiar with the Library.

In asking that such work be undertaken, members should specify clearly the subject to be covered, and whether references to general books only are desired, or whether a complete bibliography, involving search through periodical literature, is desired.

In reference to this work, the Appendices* to the Annual Reports of the Board of Direction for the years ending December 31st, 1906, and December 31st, 1910, contain summaries of all searches made to date.

PAPERS AND DISCUSSIONS

Members and others who take part in the oral discussions of the papers presented are urged to revise their remarks promptly. Written communications from those who cannot attend the meetings should be sent in at the earliest possible date after the issue of a paper in *Proceedings*.

* *Proceedings*, Vol. XXXIII, p. 20 (January, 1907); Vol. XXXVII, p. 28 (January, 1911).

All papers accepted by the Publication Committee are classified by the Committee with respect to their availability for discussion at meetings.

Papers which, from their general nature, appear to be of a character suitable for oral discussion, will be published as heretofore in *Proceedings,* and set down for presentation to a future meeting of the Society, and, on these oral discussions, as well as written communications, will be solicited.

All papers which do not come under this heading, that is to say, those which from their mathematical or technical nature, in the opinion of the Committee, are not adapted to oral discussion, will not be scheduled for presentation to any meeting. Such papers will be published in *Proceedings* in the same manner as those which are to be presented at meetings, but written discussions, only, will be requested for subsequent publication in *Proceedings* and with the paper in the volumes of *Transactions.*

LOCAL ASSOCIATIONS OF MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

San Francisco Association

The San Francisco Association of Members of the American Society of Civil Engineers holds regular bi-monthly meetings, with banquet, and weekly informal luncheons. The former are held at 6 P. M., at the Palace Hotel on the third Friday of February, April, June, August, October, and December, the last being the Annual Meeting of the Association.

Informal luncheons are held at 12.15 P. M. every Wednesday, and the place of meeting may be ascertained by communicating with the Secretary of the Association, E. T. Thurston, Jr., M. Am. Soc. C. E., 713 Mechanics' Institute, 57 Post Street.

The by-laws of the Association provide for the extension of hospitality to any member of the Society who may be temporarily in San Francisco, and any such member will be gladly welcomed as a guest.

Colorado Association

The meetings of the Colorado Association of Members of the American Society of Civil Engineers are held on the second Saturday of each month, except July and August. The hour and place of meeting are not fixed, but this information will be furnished on application to the Secretary, Gavin N. Houston, M. Am. Soc. C. E., 409 Equitable Building, Denver, Colo. The meetings are usually preceded by an informal dinner. Members of the American Society of Civil Engineers will be welcomed at these meetings.

Weekly luncheons are held on Wednesdays, and, until further notice, will take place at the Colorado Traffic Club.

Visiting members are urged to attend the meetings and luncheons.

(Abstract of Minutes of Meetings)

September 14th, 1912.—The meeting was called to order; President Ketchum in the chair; G. N. Houston, Secretary; and present, also, 20 members and 11 guests.

The subject for discussion, "The Cherry Creek Problem," was introduced by A. L. Fellows, M. Am. Soc. C. E., and was discussed by Messrs. Hunter, Bradley, Cranmer, Salter, Treise, Comstock, Prince, De Berard, and others.

Adjourned.

October 12th, 1912.—The meeting was called to order; President Ketchum in the chair; G. N. Houston, Secretary; and present, also, 18 members and 5 guests.

A paper entitled "The Construction of the Minnequa-Walsenbury Double-Track Line," was presented by A. O. Ridgway, M. Am. Soc. C. E., who illustrated his remarks with stereopticon views, and the subject was discussed generally by the members present.

Adjourned.

November 9th, 1912.—The meeting was called to order; President Ketchum in the chair; and present, also, 16 members and 4 guests.

The subject for discussion, "The Properties of Timber That Make for Durability," was opened by Mr. Norman DeW. Betts, of the United States Forestry Service, and Sam G. Porter, M. Am. Soc. C. E., who illustrated his remarks with lantern slides. A general discussion followed, in which the subject was presented from the viewpoint of the irrigation engineer by Messrs. Anderson and Ulrich; from the bridge and building standpoint by H. S. Crocker, M. Am. Soc. C. E.; from the viewpoint of the railroad engineer by Messrs. Vincent and Ridgway; and from the municipal tramway engineer's standpoint by R. W. Toll, Jun. Am. Soc. C. E.

Adjourned.

Atlanta Association

On March 14th, 1912, the Atlanta Association of Members of the American Society of Civil Engineers was organized, with the following officers: Arthur Pew, President; William A. Hansell, Jr., Secretary; and Messrs. James N. Hazlehurst and Alexander Bonnyman, Members of the Executive Committee. The Association will hold its meetings in the house of the University Club.

PRIVILEGES OF ENGINEERING SOCIETIES EXTENDED TO MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

Members of the American Society of Civil Engineers will be welcomed by the following Engineering Societies, both to the use of their Reading Rooms and at all meetings:

American Institute of Mining Engineers, 29 West Thirty-ninth Street, New York City.

American Society of Mechanical Engineers, 29 West Thirty-ninth Street, New York City.

Architekten-Verein zu Berlin, Wilhelmstrasse 92, Berlin W. 66, Germany.

Associação dos Engenheiros Civis Portuguezes, Lisbon, Portugal.

Australasian Institute of Mining Engineers, Melbourne, Victoria, Australia.

Boston Society of Civil Engineers, 715 Tremont Temple, Boston, Mass.

Brooklyn Engineers' Club, 117 Remsen Street, Brooklyn, N. Y.

Canadian Society of Civil Engineers, 413 Dorchester Street, West, Montreal, Que., Canada.

Civil Engineers' Society of St. Paul, St. Paul, Minn.

Cleveland Engineering Society, Chamber of Commerce Building, Cleveland, Ohio.

Cleveland Institute of Engineers, Middlesbrough, England.

Dansk Ingeniorforening, Amaliegade 38, Copenhagen, Denmark.

Engineers' and Architects' Club of Louisville, Ky., 303 Norton Building, Fourth and Jefferson Streets, Louisville, Ky.

Engineers' Club of Baltimore, Baltimore, Md.

Engineers' Club of Minneapolis, 17 South Sixth Street, Minneapolis, Minn.

Engineers' Club of Philadelphia, 1317 Spruce Street, Philadelphia, Pa.

Engineers' Club of St. Louis, 3817 Olive Street, St. Louis, Mo.

Engineers' Club of Toronto, 96 King Street, West, Toronto, Ont., Canada.

Engineers' Society of Northeastern Pennsylvania, 302 Board of Trade Building, Scranton, Pa.

Engineers' Society of Pennsylvania, 219 Market Street, Harrisburg, Pa.

Engineers' Society of Western Pennsylvania, 2511 Oliver Building, Pittsburgh, Pa.

Institute of Marine Engineers, 58 Romford Road, Stratford, London, E., England.

Institution of Engineers of the River Plate, Buenos Aires, Argentine Republic.

Institution of Naval Architects, 5 Adelphi Terrace, London, W. C., England.

Junior Institution of Engineers, 39 Victoria Street, Westminster, S. W., London, England.

Koninklijk Instituut van Ingenieurs, The Hague, The Netherlands.

Louisiana Engineering Society, 321 Hibernia Bank Building, New Orleans, La.

Memphis Engineering Society, Memphis, Tenn.

Midland Institute of Mining, Civil and Mechanical Engineers, Sheffield, England.

Montana Society of Engineers, Butte, Mont.

North of England Institute of Mining and Mechanical Engineers, Newcastle-upon-Tyne, England.

Oesterreichischer Ingenieur- und Architekten-Verein, Eschenbachgasse 9, Vienna, Austria.

Pacific Northwest Society of Engineers, 803 Central Building, Seattle, Wash.

Rochester Engineering Society, Rochester, N. Y.

Sachsischer Ingenieur- und Architekten-Verein, Dresden, Germany.

Sociedad Colombiana de Ingenieros, Bogota, Colombia.

Sociedad de Ingenieros del Peru, Lima, Peru.

Societe des Ingenieurs Civils de France, 19 Rue Blanche, Paris, France.

Society of Engineers, 17 Victoria Street, Westminster, S. W., London, England.

Svenska Teknologforeningen, Brunkebergstorg 18, Stockholm, Sweden.

Tekniske Forening, Vestre Boulevard 18–1, Copenhagen, Denmark.

Western Society of Engineers, 1737 Monadnock Block, Chicago, Ill.

ACCESSIONS TO THE LIBRARY

(From November 6th to December 3d, 1912)

DONATIONS*

A TEXT-BOOK ON ROADS AND PAVEMENTS.

By Frederick P. Spalding, M. Am. Soc. C. E. Fourth Edition, Revised and Enlarged. Cloth, $7\frac{1}{2}$ x 5 in., illus., 11 + 408 pp. New York, John Wiley & Sons; London, Chapman & Hall, Limited, 1912. $2.00.

In the preface to the first edition of this work, it is stated that the author's aim has been to give a brief discussion, from an engineering standpoint, of the principles involved in highway work and to outline the more important systems of construction with a view to forming a text which may serve as a basis for a systematic study of the subject. Changes in the character of traffic, due to the introduction of automobiles, and modifications of the standards of life in city and country have caused, it is stated, a more careful and scientific study of materials for, and the use of more effectual methods in, the construction and maintenance of highways, and consequently many changes have been made in the subject-matter of the book. The third edition, issued in 1908, was practically rewritten, and the present edition includes new chapters on Bituminous Macadam and Concrete Pavements and the chapters on Brick, Asphalt and Wood Pavements have been considerably modified. The Chapter headings are: Road Economics and Management; Drainage of Streets and Roads; Location of Country Roads; Improvement and Maintenance of Country Roads; Broken-Stone Roads; Bituminous Macadam Roads; Foundations for Pavements; Brick Pavements; Asphalt Pavements; Wood-Block Pavements; Stone-Block Pavements; Concrete Pavements; City Streets.

STREET PAVEMENTS AND PAVING MATERIALS

A Manual of City Pavements: The Methods and Materials of Their Construction, for the Use of Students, Engineers, and City Officials. By George W. Tillson, M. Am. Soc. C. E. Second Edition. Cloth, $9\frac{1}{4}$ x 6 in., illus., 16 + 651 pp. New York, John Wiley & Sons; London, Chapman & Hall, Limited, 1912. $4.00.

The first edition of this book was published in 1900, since which time many types of new pavements have been introduced and the methods of constructing the older types greatly modified. These changes have necessitated, it is stated, changes in the text of this work, which include a complete revision of certain chapters and the addition of a new chapter on Concrete Pavements. The chapter on The Protection of Pavements is also new, and the author hopes it will prove especially useful in cities where much subsurface street construction is necessary. As stated in the secondary title, the volume is devoted to a study of the methods and materials for constructing city pavements, the subject of road construction being entirely ignored. The Chapter headings are: The History and Development of Pavements; Stone; Asphalt; Brick-Clays and the Manufacture of Paving Brick; Cement, Cement Mortar and Concrete; The Theory of Pavements; Cobble and Stone-Block Pavements; Asphalt Pavements; Brick Pavements; Wood Pavements; Broken-Stone Pavements; Concrete Pavements; Plans and Specifications; The Construction of Street-Car Tracks in Paved Streets; Width of Streets and Roadways, Curbs, Sidewalks, etc.; Asphalt Plants; The Protection of Pavements; Index.

THE MINING WORLD INDEX

Of Current Literature, Vol. 1, 1912. By Carpel L. Breger. Cloth, $9\frac{1}{4}$ x 6 in., 31 + 317 pp. Chicago, Mining World Company, 1912.

In a secondary title it is stated that this book is an international bibliography of mining and the mining sciences, namely, mining, ore dressing, metallurgy, assaying, geology, etc. It is a compilation and revision of the classified index of the world's current literature on mining, metallurgical, and the affiliated mineral industries which has appeared weekly in *Mining and Engineering World* since January, 1911. It will be issued in semi-annual volumes, of which the present volume is the first, being for the first semester of 1912, and is intended for the use of the practical miner, the mill man, the metallurgist, the layman, the operator, the student, and the trained or expert engineer. In this volume the larger subject headings are said to have been subdivided and all the different technical journals in which an

*Unless otherwise specified, books in this list have been donated by the publishers.

article has appeared are indicated, a feature which, it is thought, will be appreciated by those whose library facilities are limited. The Contents are: I, Geography; II, Ores and Mineral Products; III, Technology; Explanations and Abbreviations; Author Index; Subject Index.

RAILWAY TRANSPORTATION

A History of its Economics and of its Relation to the State. By Charles Lee Raper. Cloth, 8 x 5¼ in., 11 + 331 pp. New York and London, G. P. Putnam's Sons, 1912. $1.50.

The author's chief purpose in this book, which is intended for the general reader as well as the special student of railways, has been to revise and enlarge Hadley's "Railroad Transportation," on which, with the author's permission, it is based. The author states that the development of railway transportation has been so great and so important in its relation to the State since President Hadley's book was written in 1885 that it is vitally important to bring the subject down to the present. He traces the history of railway transportation, it is stated, in its more vital aspects, in the United States, Great Britain, France, Italy, and Germany, solely to throw light on the present management and regulation of railways, and, with his statements, includes comparisons and estimates of values. In the final chapter the statement of the reasons and methods as well as the history of State operation in various foreign countries, is said to be important. In order to ascertain the most reliable facts in connection with the subject, the author has personally examined records and the secondary sources as well as the conditions of the lines and equipment, the methods of operation, and the general characteristics of the traffic in all the countries which have come under treatment. The Contents are: Modern Transportation; Railway Transportation in Great Britain; Railway Transportation in France; Railway Transportation in Italy; Railway Transportation in Germany; Railway Transportation in the United States; State Operation of Railways; Extension of Parcels Post, etc.; Index.

HENDRICKS' COMMERCIAL REGISTER OF THE UNITED STATES

For Buyers and Sellers. Twenty-first Annual Edition. Cloth, 10½ x 7½ in., illus., 122 + 1576 pp. New York, Samuel E. Hendricks Co., 1912. $10.00.

This volume, which is devoted to the interests of the architectural, mechanical, engineering, contracting, electrical, railroad, iron, steel, hardware, mining, mill, quarrying, exporting, and kindred industries, is stated to be a complete and reliable annual index of these industries containing more than 350 000 names and addresses and upward of 40 000 business classifications. It is said to be indispensable as a buyer's reference and for mailing purposes, and gives full lists of manufacturers of and dealers in everything used in the manufacture of material, machinery, and apparatus for these industries, from the raw material to the manufactured article and from the producer to the consumer. Its contents are arranged alphabetically by subject, under which are given, in alphabetical order and in some cases by States and cities, the names and addresses of firms dealing in a particular article, and sometimes these are followed by detailed matter, titles of identification, trade names, etc. There is also an alphabetical list of advertisers including the addresses of their domestic and foreign branches, a simplified discount sheet for the purchasing agent, and an index to contents of 122 pages.

SCIENTIFIC MANAGEMENT:

Addresses and Discussions at the First Conference at the Amos Tuck School, Dartmouth College, Held October 12th-14th, 1911. Cloth, 9¼ x 6 in., illus., 11 + 388 pp. Hanover, N. H., Dartmouth College, 1912. (Donated by Harlow S. Person). $2.75.

In an address on Scientific Management before the Social Science Club and the Dartmouth Scientific Society of Dartmouth College, Harlow S. Person, Director of the Amos Tuck School of Administration and Finance, stated that the purpose of this first Tuck School Conference was to enable business men and manufacturers of New Hampshire and of New England to meet the organizing engineers who have applied scientific management and the manufacturers in whose plants it is in operation. As stated in the title, this volume is made up of the addresses and discussions at this Conference, which, it is hoped, will aid in a better understanding of the principles of scientific management and of its applicability to various businesses. The Contents are: Introduction: Scientific Management, by Harlow S. Person. I, The Principles of Scientific Management, by Frederick W. Taylor.

II, Scientific Management and the Laborer: The Task and the Day's Work, by Henry L. Gantt; The Opportunity of Labor Under Scientific Management, by Harrington Emerson. III, Scientific Management and the Manager: Types of Management—Unsystematized, Systematized, and Scientific, by Henry P. Kendall; The Spirit in Which Scientific Management Should be Approached, by James M. Dodge. IV, Discussions on the Applicability of Scientific Management in Certain Industries: Machine Manufacture; Textile Manufacture; Shoe Manufacture; Printing and Publishing; Pulp and Paper Manufacture; Lumbering, and the Management of Timber Properties; Academic Efficiency. V, Scientific Management and Government: The Application of Scientific Management to the Activities of State and Municipal Government, by Frederick A. Cleveland. VI, Phases of Scientific Management: Symposium; Registration at the Conference.

McGRAW ELECTRIC RAILWAY MANUAL, 1912.

The Red Book of American Street Railway Investments. Edited by Frederic Nicholas. Nineteenth Annual Number. Cloth, 13 x 10 in., illus., 32 + 344 pp. New York, McGraw Publishing Company, 1912. $5.00.

This book, it is stated in a secondary title, is a manual, issued in connection with the *Electric Railway Journal*, of securities, traffic statistics, earnings, officers, directors, and equipment of street and interurban railways in the United States, Canada, Mexico, Cuba, and the West Indies. The general arrangement of the subject-matter, as given in previous issues, is followed in this volume, alphabetically by States and cities with the history, capital stock, funded debt, mortgages, track and equipment, names and addresses of officers, and addresses of the general offices and repair shops of each company. There are maps showing the main and connecting lines of many of the larger companies and, at the end of the book, is given a list of the various street and interurban railway associations with the names and addresses of their officers. The gross earnings of electric railway companies are given, as well as the changes shown by the 1912 edition of the Manual, details of operating statements, and a list of companies with gross earnings in 1911 of more than $1 000 000. There is an index of ten pages of the companies described in the Manual.

STRUCTURAL DETAILS OF HIP AND VALLEY RAFTERS.

By Carlton Thomas Bishop. Cloth, 8 x 10¾ in., illus., 5 + 72 pp. New York, John Wiley and Sons; London, Chapman and Hall, Limited, 1912. $1.75.

The author states that his purpose in this book is to present the subject of hip and valley construction so completely that any draftsman with a reasonable knowledge of structural details and of trigonometry can make working drawings which shall give all necessary information to the shop without useless refinements. No attempt has been made, it is stated, to show the application to skew portals, hoppers, or chutes, but it is felt that the fomulas will be of great assistance to draftsmen when dealing with these problems. Complete directions are given, it is said, for making shop drawings for the steelwork of intersecting roofs and similar structures, the notes for the various cases being arranged for convenient reference and illustrated by general drawings and typical problems. The algebraic and graphic methods of obtaining the necessary numerical values are fully explained, and, at the end of the book, tables are given to assist in the solution of problems which are most likely to occur in practice. The Contents are: General Outline; Flange Connection; Web Connection; Notes on Other Cases; Derivation of Formulas; Graphic Method of Determining Angles; Values and Logarithms for Common Cases.

THE DESIGN OF SIMPLE ROOF-TRUSSES IN WOOD AND STEEL

With an Introduction to the Elements of Graphic Statics. By Malverd A. Howe, M. Am. Soc. C. E. Third Edition, Revised and Enlarged. Cloth, 9¼ x 6 in., illus., 8 + 179 pp. New York, John Wiley & Sons; London, Chapman & Hall, Limited, 1912. $2.00.

In the preface to the first edition of this work, published in 1902, the author states that his object was to bring together all the essentials necessary to the proper design of ordinary roof-trusses in wood and steel, which, previous to that date, had been accessible only in the various comprehensive treatises on the subject and in manufacturers' pocket-books. In this edition considerable new matter, it is stated, will be found in the body of the text and in the Appendix. The design of details

in wood has also been revised, the standard or actual sizes of wood being used instead of the nominal sizes. The Chapter headings are: General Principles and Methods; Beams and Trusses; Strength of Materials; Roof Trusses and Their Design; Design of a Wooden Roof-Truss; Design of a Steel Roof-Truss; Tables; Appendix; Index.

AN OUTLINE OF THE METALLURGY OF IRON AND STEEL.

By A. Humboldt Sexton and J. S. G. Primrose. Second Edition. Cloth, 8¾ x 5¾ in., illus., 16 + 572 pp. Manchester, England, The Scientific Publishing Company, 1912. 12 shillings 6 pence.

This book, it is stated, was prepared to meet the need of one of the authors in his teaching, namely, a book which in one volume of moderate size would cover the whole field of the metallurgy of iron and steel. This, the second edition, has been carefully revised and some of the chapters have been rewritten in order, it is said, to bring the subject-matter up to date. All the more important developments in processes and plant are described, only such descriptions of the older processes being retained as are necessary to an understanding of modern development and historical interest. Considerable attention has been given, it is stated, in this edition to the metallography and heat treatment of the metal. Numerous references to original papers are included, and the authors urge their readers, especially students, to make a study of these papers and the methods described in them. The Contents are: Part I, Introductory; Part II, Iron; Part III, Malleable Iron; Part IV, Steel; Appendix; Index.

FOWLER'S MECHANICAL ENGINEER'S POCKET BOOK, 1913.

Edited by William H. Fowler. Fifteenth Annual Edition. Leather, 6¼ x 4 in., illus., 66 + 592 pp. Manchester, England, Scientific Publishing Co., 1912. 2 shillings 9 pence.

The Contents are: Miscellaneous Tables and Formulæ; Steam Boilers and Fittings; Fuels and Combustion; Steam Engines; Steam Turbines; Locomotives; Steam Tables; Valves and Valve Gear; Gas Engines; Gases Used in Gas Engines; Oil Engines; Hydraulics; Pumps and Pumping Arrangements; Gearing and Lubrication; Hoisting and Lifting Machinery; Mining Machinery and Appliances; Iron and Steel; Metals and Alloys; Beams and Pillars; Springs; Chemistry; Ventilation and Heating; Index.

MODERN HOSPITALS:

A Series of Authoritative Articles on Planning and Equipment, as Exemplified by the Best Practice in This Country and Europe. By Edward F. Stevens and others. Cloth, 12½ x 9¼ in., illus., 49 + 86 pp. New York, The American Architect, 1912. $5.00.

The preface states that the subject-matter of this book is descriptive of the latest word on hospital construction, arrangement, and equipment, based on the best modern practice, and that it is intended as an aid to architects and those concerned with the superintendence of hospitals and the care of the sick. The text is supplemented by many illustrations of recently constructed hospitals, for special and general fields, consisting of floor plans, elevations, perspectives, and illustrations of interiors and of technical equipments. A partial list of Contents is: Details and Equipment of Hospitals, by Edward F. Stevens; Modern Practice in Hospital Heating and Ventilation, by Clarence W. Williams; Some Essentials of Hospital Heating and Ventilation, by D. D. Kimball; Hospital Lighting, by E. H. Bostock; The Artificial Lighting of Hospitals, by John Darch; Co-operation in Hospital Planning, by M. E. McCalmont; A Tropical Hospital Adaptable for Tuberculosis, by M. E. McCalmont; Descriptions and Illustrations of the Barnard Skin and Cancer Hospital; Contagious Group of the Providence, R. I., City Hospital; Brooklyn Seaside Hospital for Children; St. Vincent's Hospital, Indianapolis, Ind.; the New Bellevue Hospital, New York City; the Rockefeller Institute for Medical Research, New York City, etc., etc.

A TREATISE ON CEMENT SPECIFICATIONS.

By Jerome Cochran, Jun. Am. Soc. C. E. Cloth, 8½ x 5½ in., illus., 12 + 101 pp. New York, D. Van Nostrand Company, 1912. $1.00.

In a secondary title it is stated that this treatise includes the general use, purchase, storage, inspection, and test requirements of Portland, natural, puzzolan (slag), and silica (sand) cements, together with methods of testing and analysis of Portland cement. The author's aim has been to present a set of specifications for cement in a form for convenient practical use and ready reference, which will be

consistent and conform to modern practice. In order to enable the student or young engineer to study the methods used by others in drawing up cement specifications, the author has included numerous but carefully selected references to specification requirements for cement contained in engineering periodicals and the transactions of engineering societies. The Contents are: Introduction; General Conditions Governing Use of Cement; Furnishing Cement to the Contractor; Purchase of Cement from Manufacturers; Delivery and Storage of Cement; Inspection and Tests of Cement; Test Requirement for Cement; Methods of Testing Cement; Significance of Tests of Cement; Methods of Chemical Analysis of Portland Cement; Bibliography of Specifications for Cement; Bibliography of Foreign Cement Specifications; Index.

Gifts have also been received from the following:

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Woburn, Mass.-City Clerk. 1 bound vol.

BY PURCHASE

Mitteilungen über Forschungsarbeiten auf dem Gebiete des Ingenieurwesens, insbesondere aus den Laboratorien der technischen Hochschulen. Herausgegeben vom Verein deutscher Ingenieure. Hefte 121–124. Julius Springer, Berlin, 1911.

Smoke: A Study of Town Air. By Julius B. Cohen and Arthur G. Ruston. Edward Arnold, London, 1912.

Historical Papers on Modern Explosives. By George W. MacDonald, With an Introduction by *Sir* Andrew Noble. Whittaker & Co., New York and London, 1912.

The Electric Circuit. By V. Karopetoff. Second Edition. McGraw-Hill Book Co., New York and London, 1912.

Skeleton Construction in Buildings; With Numerous Practical Illustrations of High Buildings. By William H. Birkmire. Fourth Edition. John Wiley & Sons, New York; Chapman & Hall, Ltd., London, 1907.

Methods of Air Analysis. By J. S. Haldane. Charles Griffin & Co., Ltd., London, 1912.

The Mechanical Engineering of Collieries. By T. Campbell Futers. Vol. 2, Revised and Enlarged. The Colliery Guardian Co., Ltd., London, 1910.

Vorlesungen über Ingenieur-Wissenschaften. Von Georg Christoph Mehrtens. Erster Teil. Statik und Festigkeitstlehre. Dritter Band, Zweite Hälfte. Wilhelm Engelmann, Leipzig, 1912.

Modern Brickmaking. By Alfred B. Searle. D. Van Nostrand Co., New York; Scott, Greenwood & Son, London, 1911.

SUMMARY OF ACCESSIONS

(From November 6th to December 3d, 1912)

Donations (including 41 duplicates)	564
By purchase	12
Total	576

MEMBERSHIP

ADDITIONS

(From November 8th to December 5th, 1912)

MEMBERS

		Date of Membership.
BEAN, GEORGE LEWIS. Hydr. Engr., 1729 North 19th St., Philadelphia, Pa.		Oct. 1, 1912
GAILLARD, JAMES JOSIAS. City Engr., City Hall, Macon, Ga.		Oct. 1, 1912
MATTIS, GEORGE. Prin. Asst. Engr., Div. V, California Highway Comm., San Luis Obispo, Cal.		Oct. 29, 1912
PALMER, GEORGE BRUCE. Const. Engr., Michigan Alkali Co., Wyandotte, Mich.		Sept. 3, 1912
SAPH, AUGUSTUS VALENTINE. 2330 Durant Ave., Berkeley, Cal.	Assoc.	Oct. 1, 1901
	Assoc. M.	June 4, 1907
	M.	Oct 29, 1912
SUTTON, CHARLES WOOD. Chf. Engr., Peruvian Irrig. Service, Apartado 889, Lima, Peru	Jun.	Dec. 1, 1903
	Assoc. M.	Dec. 6, 1905
	M.	Sept. 3, 1912
WHITTEMORE, JOSEPH OGIER. 116 West Sidney Ave., Mt. Vernon, N. Y.		Oct. 29, 1912

ASSOCIATE MEMBERS

ANDERSON, LOWREY WALLACE. Gen. Mgr. and Chf. Engr., Pecos Val. South. Ry., Pecos, Tex.	Jun.	April 6, 1909
	Assoc. M.	Oct. 29, 1912
ANDREWS, CARL BOWERS. Chf. Engr., Oahu Ry., 743 Wyllie St., Honolulu, Hawaii		Oct. 29, 1912
BEATY, ROBERT ERNEST. 210 West 107th St., New York City		Oct. 1, 1912
BOYD, JOSEPH CHARLES. Civ. Engr. and Surv., 1007 Eighth St., Sacramento, Cal.		Oct. 29, 1912
CATE, DANIEL ROGERS. (Phinney, Cate & Marshall), Room 420, Forum Bldg., Sacramento, Cal.		July 9, 1912
ELLINGSON, OLAF JOHN SVERDROP. Contr. Engr., Midland Bridge Co. of Kansas City, Mo., 406 South Crockett St., Sherman, Tex.		Oct. 29, 1912
FLAA, INGWALD EDWARD. Designing Engr., Spring Val. Water Co., 375 Sutter St., San Francisco, Cal.		Oct. 29, 1912
FREEMAN, WILLIAM BRADLY. Care, Dept. of Ways of Communication, Bangkok, Siam		Dec. 3, 1912
GOODRICH, CLINTON RAYMOND. Supt., James Stewart & Co., First National Bank Bldg., Houston, Tex.		Oct. 29, 1912
GRAHAM, JOHN WILLIAM. Dist. Engr., Bureau of Public Works, Manila, Philippine Islands	Jun.	Oct. 4, 1910
	Assoc. M.	Sept. 3, 1912
GREEN, HARRY EDGAR. City Engr., Waterville, Me.		July 9, 1912

ASSOCIATE MEMBERS (*Continued*)		Date of Membership.
HANMER, HARRY J. Asst. City Engr., 79 Broad St., Gloversville, N. Y.		Oct. 1, 1912
HINCKLEY, GEORGE STEVENS. City Engr. and Supt. of Streets, Redlands, Cal.		Oct. 1, 1912
HOGAN, JOSEPH VINCENT. Res. Engr., Dept. of State Engr. and Surv., Barge Canal Office, Medina, N. Y.		Oct. 29, 1912
HOLLAND, CLIFFORD MILBURN. Asst. Engr., Public Service Comm., First Dist. (Res., 933 East 22d St.), Brooklyn, N. Y.		Oct. 29, 1912
HORTENSTINE, RALEIGH. Contr. Engr., Virginia Bridge Co. of Texas, P. O. Box 956, Dallas, Tex.		Oct. 29, 1912
KEPPEL, PAUL HENRY. Asst. Engr., Cuban Central Rys., Ltd., Sagua la Grande, Cuba		Oct. 29, 1912
MACKAY, ANGUS ROBERT. Mgr., Vulture Mines Co., Wickenburg, Ariz.		Oct. 29, 1912
MEYER, GROVER JOHN. Asst. Engr., Sultan River Hydro-Elec. Project, Sultan, Wash.		Oct. 29, 1912
MITTMANN, EGMONT FELIX. Care, Am. Rio Grande Land & Irrig. Co., Mercedes, Tex.		Oct. 1, 1912
OGDEN, HAROLD COE. Asst. Engr., The Arkansas Val. Sugar Beet & Irrigated Land Co., Box 180, Holly, Colo.		Oct. 29, 1912
PAGE, EDWIN RANDOLPH. Min. Engr., The Gauley Mountain Coal Co., Jodie, W. Va.		Oct. 1, 1912
PEABODY, GEORGE ALFRED. Engr. and Asst. to Supt., Cleveland Frog & Crossing Co., 1436 East 111th St., N. E., Cleveland, Ohio		Oct. 29, 1912
PHELPS, TRACY IRWIN. Res. Engr., U. S. Reclamation Service, Thistle, Utah		Oct. 29, 1912
PLANT, FRANCIS BENJAMIN. 922 Rialto Bldg., San Francisco, Cal.		Oct. 29, 1912
SAWHNEY, ASA NAND. Engr., Kashmere State Palaces, Jumma (Tawi), India		Sept. 3, 1912
SHERMAN, HENRY ANDREW. Junior Engr., U. S. Engr. Dept., 309 Armory Pl., Sault Ste. Marie, Mich.		Oct. 29, 1912
STILSON, CHARLES EDWARD. 336 Brunswick Ave., Toronto, Ont., Canada	Jun. Assoc. M.	Mar. 1, 1910 Oct. 29, 1912
STINE, WALTER PEARCE. 705 Elgie St., Beaumont, Tex.		Oct. 1, 1912
WARNOCK, WILLIAM HAROLD. Asst. Engr., Board of Water Supply, 601 West 149th St., New York City	Jun. Assoc. M.	April 4, 1911 Oct. 29, 1912

JUNIORS

BAILEY, RUSSEL THOMAS. Res. Engr., Ambursen Hydr. Constr. Co., Branson, Mo.		May 28, 1912
BOERNER, FRANCIS CLARENCE. Care, Turner Constr. Co., 11 Broadway (Res., 228 Edgecombe Ave.,) New York City		Oct. 29, 1912

JUNIORS (*Continued*)	Date of Membership.
CHASE, CLEMENT EDWARDS. Asst. Engr., Cherry St. Bridge, 510 Michigan Apartments, Toledo, Ohio....	Oct. 1, 1912
HORWEGE, ALVIN ARTHUR. Asst. Engr., State Board Harbor Commrs., 1418 Larkin St., San Francisco, Cal...	Oct. 29, 1912
KELLY, HUGH AMBROSE. Engr. and Asst. Secy., City Plan Comm., 33 Baldwin Ave., Jersey City, N. J.........	Oct. 29, 1912
PAGE, PERCY HAROLD. Senior Draftsman, Rees & Kirby, Ltd., Morriston, near Swansea, Wales.............	May 28, 1912
PATTERSON, CHARLES SCOTT. Asst. Engr., M., K. & T. of T. Ry., Greenville, Tex..............................	Oct. 29, 1912
SHAW, GUY RAY. 810 Observatory Bldg., Des Moines, Iowa.	May 28, 1912
WILLIAMS, FREDERICK. Surv. and Draftsman, Care, U. S. Engr. Office, New London, Conn..................	Oct. 29, 1912

DEATHS

BRINSMADE, DANIEL SEYMOUR. Elected Member, February 1st, 1888; died September 7th, 1912.

KIMBALL, GEORGE ALBERT. (*Director.*) Elected Junior, May 12th, 1875; Member, July 1st, 1891; died December 3d, 1912.

Total Membership of the Society, December 5th, 1912, 6 781

MONTHLY LIST OF RECENT ENGINEERING ARTICLES OF INTEREST

(November 6th to December 4th, 1912)

NOTE.—*This list is published for the purpose of placing before the members of this Society, the titles of current engineering articles, which can be referred to in any available engineering library, or can be procured by addressing the publication directly, the address and price being given wherever possible.*

LIST OF PUBLICATIONS

In the subjoined list of articles, references are given by the number prefixed to each journal in this list:

(1) *Journal,* Assoc. Eng. Soc., Boston, Mass., 30c.
(2) *Proceedings,* Engrs. Club of Phila., Philadelphia, Pa.
(3) *Journal,* Franklin Inst., Philadelphia, Pa., 50c.
(4) *Journal,* Western Soc. of Engrs., Chicago, Ill., 50c.
(5) *Transactions,* Can. Soc. C. E., Montreal, Que., Canada.
(6) *School of Mines Quarterly,* Columbia Univ., New York City, 50c.
(7) *Gesundheits Ingenieur,* München, Germany.
(8) *Stevens Institute Indicator,* Hoboken, N. J., 50c.
(9) *Engineering Magazine,* New York City, 25c.
(10) *Cassier's Magazine,* New York City, 25c.
(11) *Engineering* (London), W. H. Wiley, New York City, 25c.
(12) *The Engineer* (London), International News Co., New York City, 35c.
(13) *Engineering News,* New York City, 15c.
(14) *Engineering Record,* New York City, 10c.
(15) *Railway Age Gazette,* New York City, 15c.
(16) *Engineering and Mining Journal,* New York City, 15c.
(17) *Electric Railway Journal,* New York City, 10c.
(18) *Railway and Engineering Review,* Chicago, Ill., 15c.
(19) *Scientific American Supplement,* New York City, 10c.
(20) *Iron Age,* New York City, 20c.
(21) *Railway Engineer,* London, England, 1s. 2d.
(22) *Iron and Coal Trades Review,* London, England, 6d.
(23) *Bulletin,* American Iron and Steel Assoc., Philadelphia, Pa.
(24) *American Gas Light Journal,* New York City, 10c.
(25) *American Engineer,* New York City, 20c.
(26) *Electrical Review,* London, England, 4d.
(27) *Electrical World,* New York City, 10c.
(28) *Journal,* New England Water-Works Assoc., Boston, Mass., $1.
(29) *Journal,* Royal Society of Arts, London, England, 6d.
(30) *Annales des Travaux Publics de Belgique,* Brussels, Belgium, 4 fr.
(31) *Annales* de l'Assoc. des Ing. Sortis des Ecoles Spéciales de Gand, Brussels, Belgium, 4 fr.
(32) *Mémoires et Compte Rendu des Travaux,* Soc. Ing. Civ. de France, Paris, France.
(33) *Le Génie Civil,* Paris, France, 1 fr.
(34) *Portefeuille Economiques des Machines,* Paris, France.
(35) *Nouvelles Annales de la Construction,* Paris, France.
(36) *Cornell Civil Engineer,* Ithaca, N. Y.
(37) *Revue de Mécanique,* Paris, France.
(38) *Revue Générale des Chemins de Fer et des Tramways,* Paris, France.
(39) *Technisches Gemeindeblatt,* Berlin, Germany, 0, 70m.
(40) *Zentralblatt der Bauverwaltung,* Berlin, Germany, 60 pfg.
(41) *Elektrotechnische Zeitschrift,* Berlin, Germany.
(42) *Proceedings,* Am. Inst. Elec. Engrs., New York City, $1.
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(50) *Stahl und Eisen,* Düsseldorf, Germany.
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PAPERS AND DISCUSSIONS

A BRIEF DESCRIPTION OF A MODERN STREET RAILWAY TRACK CONSTRUCTION.

By A. C. Polk, Assoc. M. Am. Soc. C. E.

To be Presented October 16th, 1912.

With the increasing demands made on various traction companies throughout the country to provide good service, the matter of securing a permanent roadbed in the busy and paved sections of a city has become a serious problem. When the work is poorly done originally, and is not on a proper foundation, the cost of maintenance is increased, the paving along the poor roadbed becomes cracked and ruined by the movement of the track, unfavorable public comment on it arises, and the company finds itself face to face with one of two propositions, either the entire rebuilding of the roadbed or the attempt to surface and repair the old one. Either alternative is expensive, as the work will have to be done on a busy street and will interfere with car schedules, with consequent loss of revenue to the company and inconvenience to the public. Therefore, when the paving is done originally, the permanent roadbed should be substantial and effective.

In this paper the writer has attempted to give a general description of the methods and materials used at Springfield, Mo., in rebuild-

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

ing the Springfield Traction Company's lines on streets which were being paved by the City during 1911.

The Company's franchise stipulates that it shall pave the space between its rails and for 2 ft. outside of the rails, when the City paves on the various streets occupied.

The work was done by the Company, under the writer's supervision, and aggregated more than 3 miles of new tracks.

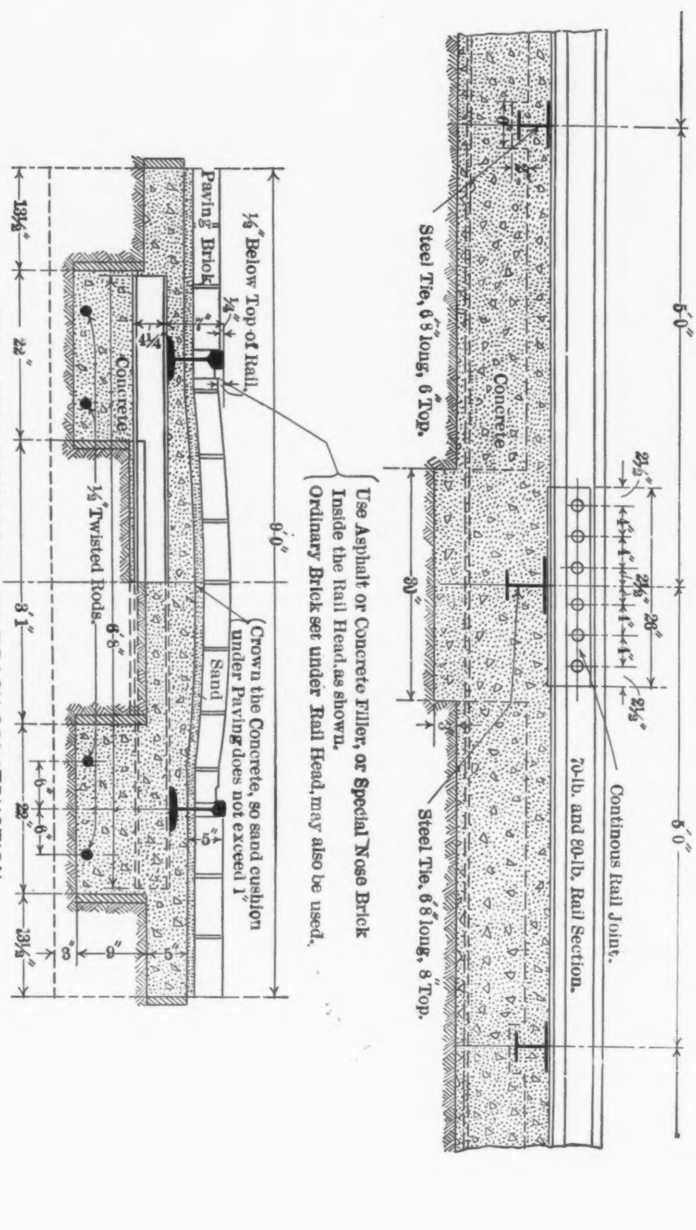
It was desired to secure as permanent a roadbed as possible, in fact, one which would outwear the rail itself and allow its replacement without disturbing the roadbed if so desired. Fig. 1 shows the general cross-section and detail of the work as put in.

General Features.—The general features of the structure were, a 7-in., 70-lb. T-rail section, 62 ft. long, laid on steel ties, and, directly under each rail, longitudinal concrete beams, each reinforced with two ½-in. twisted steel rods, and a 5-in. concrete mat over the beams and the center of the tracks for a paving foundation.

In this case the city contractors were allowed to place the street pavement first, except where it was of asphalt. After the paving on each side of the track had been completed, the old track was moved to one side or the other on the completed pavement, and excavation between the tracks was commenced, as shown on Plate XLVII.

Excavation.—The standard depth below the finished pavement to which the general excavation was made over the whole 9-ft. space was 10 in., as shown by Fig. 1. Then two trenches were excavated to exact dimensions, a depth of 9 in. more, and running with the rail. At the joints these trenches were connected by cross trenches, 30 in. wide, at an additional depth of 3 in., so that each joint tie would be thoroughly embedded and have a good thickness of concrete under it. Around each intermediate tie shallow trenches were excavated so that there would be at least 2 in. of concrete under the tie and 4 in. around its sides.

Ties.—All ties were punched at the proper gauge to take 7-in., 70-lb. or 80-lb. T-rails, but the 70-lb. rail was used principally. There were two types of ties, one under the joints which came opposite one another, and the other intermediate between them. The joint tie was an I-beam section, weighing 20 lb. per ft., 6 ft. 8 in. long, having an 8½-in. top, on which the joint rested, and a 4½-in. base. The intermediate ties were of the same length, but weighed 14.5 lb. per ft. and



SECTION CONCRETE STEEL TIE TRACK CONSTRUCTION.

Fig. 1.

had a 6-in. top and a 4-in. base. All ties were spaced 5 ft. apart, or as near that as the distance between joints would permit.

Fastenings.—The rail was fastened to the tie by special lugs and T-headed bolts with square shoulders next to the head. These bolts could be inserted from the top, if so desired, being dropped through the rectangular hole punched in the tie, turned, and then raised until the square shoulder fitted up into the rectangular hole, which prevented it from turning while running down the nuts. The lug had a projection on its under side which also fitted into the rectangular hole and prevented it from turning or backing off the rail. A socket wrench was used to run down these bolts rapidly. The joint ties were heavier and wider than the intermediate ones, as noted above, and were punched to allow the lug to fasten over the continuous joint which was used.

Joints.—Six-bolt continuous joints were used, and the following method of making them was adhered to with greatest care, as the joint is generally the weak part of the track work:

First, those parts of the rail with which the joint came in contact were carefully cleaned and polished with files and emery cloth until all scale, rust, and particles of dirt were removed and the surface was bright. Then those parts of the joints which came in contact with the rail received the same treatment. All contact parts were then greased and the joint put on. The tightening of the bolts was started from the center, working out toward the ends, pulling up a bolt on one side of the center and then the corresponding bolt on the other side, and when the bolts were all apparently tight a heavy sledge was held against the bottom of the joint on one side and the other side tapped briskly with a light sledge at the bottom only, and *vice versa.*

Then the tightening process was repeated until nothing more could be gotten out of the bolts by one man with a wrench having a 30-in. handle. After a continuous joint has once been carefully made in this manner it is there to stay, and even if the bolts were removed the joint would remain under traffic for a long period without any signs of loosening. Another feature in making a joint of this type in this manner is that no bonds were used, or considered necessary. The net area in contact has a far greater current-carrying capacity than the No. 0000 copper bond generally used on heavy lines, and a test showed no trouble with return circuits. It has also been brought to the

FIG. 1.—STREET AS LEFT BY CITY PAVING CONTRACTOR READY FOR STARTING WORK ON CAR TRACK.

FIG. 2.—OLD TRACK THROWN OVER ON NEW PAVEMENT. EXCAVATION STARTED.

writer's attention that a section of track put down in a similar manner showed a first-class negative return circuit after two years under traffic.

Surfacing and Preparing for Concrete.—The track, having been put together as described, was supported on small piers of old brick placed under every other tie, and centering under the rail. The final surfacing was done with oak wedges on top of these piers, and both the piers and wedges were concreted in.

Two ½-in. twisted steel rods were placed in the bottoms of the trenches about 6 in. apart. These also were supported at intervals on top of a brick which held them about 2½ in. off the ground. Under each joint two extra 4-ft. lengths of these steel rods were placed, to give additional strength at this point, which is usually considered the weak spot of track construction. This is shown on Fig. 1, Plate XLVIII.

The track was then given its first surfacing raise, being brought approximately to grade. Afterward, a second finishing surface and true grade was secured by driving up the oak wedges. This final surface, however, was only kept a short distance ahead of the mixer on account of the difficulty in keeping the skeleton track in correct line, variations in temperature kinking it.

Concrete.—Concreting was next started, two types of mixers being used on this particular work at different times: One was mounted on car trucks and driven with a 10 h.p. motor; it occupied the temporary old tracks which had been previously placed at one side, and was supplied by a regular work train which delivered the concrete material on flat cars. The other type was mounted on wide-tired wheels, driven with a 15 h.p., 500-volt, D. C. motor, and was equipped with a charging bucket. All materials for the latter were hauled in wagons to the streets and dumped ahead of the mixer in such manner that they were about in the right proportions. These materials were cleaned up as the mixer progressed.

Of the two methods, the latter was found to be by far the most satisfactory, as there were frequent delays in the delivery of materials to the first mixer by the work train, and the temporary track was blocked for traffic beyond the point where concreting was in progress and, on single-track work, passengers had to transfer around the obstruction.

In the second method, all material could be delivered on the street for a considerable distance ahead of the mixer, and there was no reason for the mixer to be idle at any time on this account.

The concrete was composed of 1½-in. (maximum size) crushed limestone, Kaw River sand, and chats from the Aurora Zinc Mine district, the proportions being one of cement, two of sand, two of chats, and three of crushed stone, this making a very good mixture. The concrete was mixed very wet, dumped into movable chutes from the mixer, and deposited directly by them on the track. Great care was taken at all times to keep all loose material in the excavation cleaned up ahead of the concrete and to maintain the twisted steel rods in their proper positions. The concrete was puddled carefully under and around all ties with shovels and finally crowned, when it reached grade, with a special board or templet made for the purpose.

When the material through which excavation had been made was of a very loose character, or if bad weather had caved in the sides of the trench, forms were placed where necessary to hold the concrete to the exact dimensions of the roadbed. After the concrete had set for 48 hours, and in the majority of cases longer, the brick paving was done.

Paving.—The paving was of first-class paving blocks, laid on a 1-in. sand cushion, and grouted. The method of laying it differed somewhat from the usual practice. Along each rail, and snug up against it, was laid a 2 by 4-in. piece of timber. A stretcher course of bricks was then laid against it paralleling the rail, the tops of the bricks being only ½ in. below the top of the rail. The headers across the tracks were then laid and all the bricks were thoroughly rolled and rammed.

Before grouting, the 2 by 4-in. piece was removed and the space thus left was filled with concrete to a plane 1 in. below the top of the rail, forming the flangeway for the wheels. After this had set, the whole pavement was grouted thoroughly and carefully. When filled up, the grout remaining over the top of the pavement was allowed to stand until it had attained a set and was fairly stiff, then dry sand was sprinkled over it and the whole was swept thoroughly with wire brooms, thus removing all surplus grout sticking to the tops of the bricks and presenting a very neat, clean surface. It was noticeable, too, how very much the paving contractor improved in the character

FIG. 1.—REINFORCEMENT AT JOINTS, AND GENERAL CONSTRUCTION.

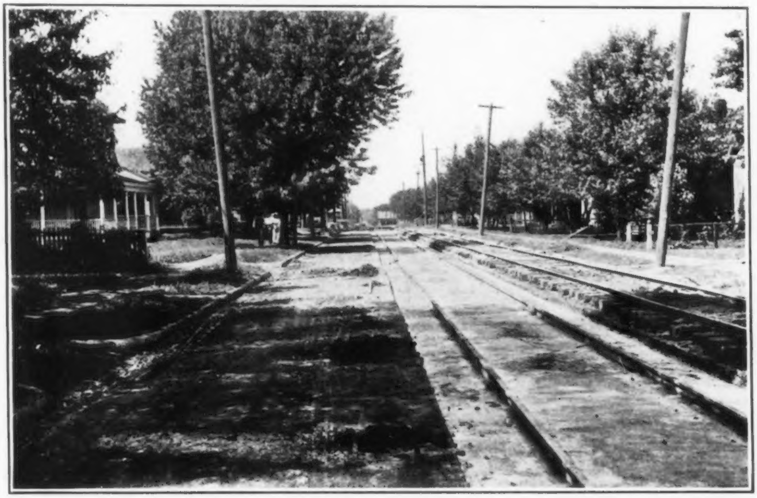


FIG. 2.—CONCRETE AND TRACK WORK COMPLETE, READY FOR BRICK PAVING.

of his work, when this specification of cleaning off the top of the brickwork was strictly enforced. By this method all irregularities of laying and surfacing are strongly brought out, and easily seen and corrected.

Cost.—The cost of track construction of this type for paved streets, including the paving between and for 2 ft. outside of the rails, using the 70-lb. high T-rail, under the conditions at Springfield, was approximately $5 per ft. of single track. This cost may seem to be high, but, when the permanent character of the work is taken into consideration, and the cost of doing street work under other methods is analyzed and compared, it will be found to be very reasonable. It possesses several advantages over some of the methods in vogue. It takes less concrete than the old method of filling under and around wooden ties. It places the concrete and the strength where it is most needed, and, when properly put in, it is certainly permanent, and the roadbed will easily outwear two or more sets of rails under heavy traffic. The rail can be changed without disturbing the roadbed in the least, if proper provisions have been made in the steel ties originally.

On one section of similar track construction, known to the writer, not the slightest sign of any motion could be detected after 3 years' operation, and it is reported to be in as good condition as the day it was built.

The writer trusts that the foregoing brief description may prove of interest to members of this Society and bring out some discussion on work of this class, concerning which very little has appeared in the technical press.

The work was done by the Springfield Traction Company, owned and operated by the Federal Light and Traction Company, of New York, and was executed under the general direction of Mr. W. A. Haller, Chief Engineer, the writer being in immediate and full charge of all construction matters at Springfield, Mo.

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NOTES ON BRIDGEWORK.*

By S. Vilar y Boy, Assoc. M. Am. Soc. C. E.

In submitting the following, the writer does not pretend to have solved any special points in engineering science. The paper contains only a re-statement of some of the problems which every-day practice has made of interest to him in the construction and maintenance of the Guatemala Railway, and this is the only reason for thinking that they may be of interest to those engaged in similar work.

For instance, the increase of the reaction on the intermediate support of a continuous beam for a double span is generally disregarded when computing trestlework; but, at the same time, and for the sake of stiffness, the stringers for trestlework are generally made continuous over two spans; therefore, the writer has considered it worth while to undertake the solution of this problem, in order to ascertain whether or not the general practice is approximate enough.

Of course, if such stringers are laid with broken joints, the total load on each bent is the same, but the partial reaction due to the stringer which is continuous on the bent considered is higher than that due to its fellow, therefore the load on such a bent is not symmetrical, and, in consequence, the foundations, whether mud-sills or piles, are likely to settle unevenly.

It is stated in the textbooks on applied mechanics that a continuous beam on three supports can be dealt with as two separate beams fixed at the intermediate point and simply supported at the ends.

*This paper will not be presented at any meeting, but written communications on the subject are invited for publication with it in *Transactions*.

In this assumption the writer has considered a concentrated load on a beam supported at one end and fixed at the other, and has worked out the solution of this problem in accordance with the general principles of the elastic theory, as follows:

If the beam be simply supported at the two ends, the reaction on A should be: $\frac{W\,b}{a+b}$, and the reaction on $B = \frac{W\,a}{a+b}$; but the effect

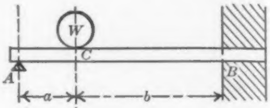


Fig. 1.

of the embedding or fixed support at B (Fig. 1) increases the reaction at this point and diminishes it at A, so that the reactions are:

$$\text{On } A \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad X = \frac{W\,b}{a+b} - Z$$

$$\text{On } B \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad Y = \frac{W\,a}{a+b} + Z.$$

Note that the increase and decrease of the reactions on the two ends must be equal, so as to fulfill the condition, $X + Y = W$; that is to say, the sum of the two reactions must be equal to the total load.

The bending moment at C, referred to the support, A, is:

$$m = \left(\frac{W\,b}{a+b} - Z\right) a$$

and the bending moment, referred to B, is:

$$m' = \left(\frac{W\,a}{a+b} + Z\right) b - M,$$

M being the bending moment at the fixed end, B.

These two moments must be equal, as corresponding to the same section, C, so that:

$$\left(\frac{W\,b}{a+b} - Z\right) a = \left(\frac{W\,a}{a+b} + Z\right) b - M; \text{ from which,}$$

$$(a + b)\,Z = M \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (1)$$

Otherwise, the general equations of the neutral fiber applied on the segment, $A\,C$, should be:

$$E\,I\,\frac{d^2\,y}{d\,x^2} = \frac{W\,b}{a+b}\,x - Z\,x$$

$$E\,I\,\frac{d\,y}{d\,x} = \frac{W\,b}{a+b}\,\frac{x^2}{2} - Z\,\frac{x^2}{2} + C$$

$$E\,I\,y = \frac{W\,b}{a+b}\,\frac{x^3}{6} - Z\,\frac{x^3}{6} + C\,x$$

C being the unknown value of the constant in the first integral, the constant of the second integral being $= 0$, as representing the value of the deflection at the support, A.

The neutral fiber equations between C and B are:

$$E\,I \frac{d^2 y'}{d\,x'^2} = \frac{W\,a}{a+b} x' + Z\,x' - M$$

$$E\,I \frac{d\,y'}{d\,x'} = \frac{W\,a}{a+b} \frac{x'^2}{2} + Z \frac{x'^2}{2} - M\,x'$$

$$E\,I\,y' = \frac{W\,a}{a+b} \frac{x'^3}{6} + Z \frac{x'^3}{6} - M \frac{x'^2}{2}$$

the constants in the two integrals being $= 0$ because B is a fixed end.

At the point, C, the two segments, $A\,C$ and $B\,C$, must be tangent to each other, and, therefore if $x = a$ in the first system of equations, and $x' = b$ in the second one, the two following conditions must be fulfilled:

$$y = y'; \quad \frac{d\,y}{d\,x} = -\frac{d\,y'}{d\,x'}$$

Therefore, the two conditions wanted are:

$$\frac{W\,b}{a+b} \frac{a^2}{2} - Z \frac{a^2}{2} + C = -\left(\frac{W\,a}{a+b} \frac{b^2}{2} + Z \frac{b^2}{2} - M\,b\right)$$

$$\frac{W\,b}{a+b} \frac{a^3}{6} - Z \frac{a^3}{6} + C\,a = \frac{W\,a}{a+b} \frac{b^3}{6} + Z \frac{b^3}{6} - M \frac{b^2}{2}$$

which can be reduced to:

$$\frac{W\,a\,b}{2} - Z \frac{a^2 - b^2}{2} = M\,b - C$$

$$\frac{W\,a\,b}{6} (a - b) - Z \frac{a^3 + b^3}{6} = -M\,b^2 - C\,a$$

The foregoing equations, together with Equation 1, will give the system:

$$(a + b)\,Z = M$$

$$\frac{W\,a\,b}{2} - Z \frac{a^2 - b^2}{2} = M\,b - C$$

$$\frac{W\,a\,b}{6} (a - b) - Z \frac{a^3 + b^3}{6} = -M \frac{b^2}{2} - C\,a$$

from which, after substituting for M its value as in Equation 1, and eliminating C, results:

$$Z = \frac{W\,a\,b\,(2\,a + b)}{2\,(a + b)^3},$$

and therefore
$$M = \frac{W\,a\,b\,(2\,a + b)}{2\,(a + b)^2}.$$

Call the total span l and:

$$Z = \frac{W\,a\,b\,(l + a)}{2\,l^3};\ M = \frac{W\,a\,b\,(l + a)}{2\,l^2}.$$

The following is an application of the foregoing theory to the "Mogul" engines of the Guatemala Railway (3-ft. gauge) on the pile bridges of that road:

According to the actual conditions, the loading diagram for the maximum reaction on the intermediate support is as shown in Fig. 2.

The total load on B is equal to the sum of the two reactions due to the two 11-ton driver axles plus the 12-ton axle which bears directly on the support.

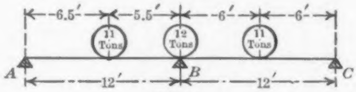


FIG. 2.

The reaction on B due to the left-hand span

$$= \frac{W\,a}{l} + \frac{W\,a\,b\,(l + a)}{2\,l^3} = 5.96 + 2.10 = \ldots 8.06 \text{ tons.}$$

The reaction on B due to the right-hand span

$$= \frac{W'\,a'}{l} + \frac{W'\,a'\,b'\,(l + a')}{2\,l^3} = 5.5 + 2.06 = 7.56 \text{ ''}$$

Total......................	15.62 tons.
Axle bearing directly on B.........	12.00 "
Total reaction on B.........	27.62 tons.

Whereas, disregarding the continuity of the beam, the total reaction on the same point should be 23.46 tons.

Therefore, the increase due to the continuity is 18 per cent.

As for the bending moment, it is not worth while to take into consideration the continuity of stringers in order to calculate their size, because their strength is governed somewhat by the horizontal shearing along the neutral fiber, and the equation of moments for

the continuous beams would give too small a size to resist the horizontal shearing.

The following example will illustrate this:

With the same "Mogul" engine, the loading diagram for the maximum bending moment is as shown by Fig. 3.

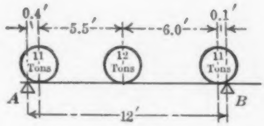


FIG. 3.

Disregarding the continuity:

The reaction at A	=	16.8	tons.
The reaction at B	=	17.2	tons.
The maximum bending moment	=	38.7	ft-tons.
The maximum bending moment under each rail	=	19.3	ft-tons.
" " " " " " "	=	232	in-tons.
The maximum shear under each rail	=	8.60	tons.

Assuming the cross-bending stress = 1 000 lb. per sq. in., that is, 0.5 tons, $\frac{I}{Z}$ (section modulus) = 464, which practically corresponds to two 6 by 16-in. pieces.

Continuous Beam.—

Left-hand axle	$Z = 0.18$ tons;	$M = 2.16$ ft-tons.
Middle axle	$Z = 2.23$ "	$M = 26.76$ "
Right-hand axle	$Z = 0.09$ "	$M = 1.08$ "
Totals	$Z = 2.50$ tons;	$M = 30.00$ ft-tons.

Therefore, the total reaction at $B = 17.2 + 2.5 = 19.7$ tons.

The maximum shear under each rail = 9.85 tons.

The maximum bending moment under each rail = 15.00 ft-tons = 180 in-tons.

If the cross-bending stress = 0.5 tons, $\frac{I}{Z} = 360$, which corresponds to two 4 by 16-in. pieces.

Longitudinal Shearing Strain.—

$$H = \frac{3}{2} \frac{\text{Maximum vertical shear}}{\text{Cross-section of beam}}$$

For two pieces 6 by 16 in. (disregarding continuity)

$$H = \frac{3}{2} \frac{8.6}{12 \times 16} = 0.067 \text{ tons} = 134 \text{ lb. per sq. in.}$$

For two pieces 4 by 16 in. (continuous beam)

$$H = \frac{3}{2} \frac{9.85}{8 \times 16} = 0.115 \text{ tons} = 230 \text{ lb. per sq. in.}$$

Whereas, the actual reaction, 9.85 tons, due to the continuous beam, applied to the size deduced from the bending moment, disregarding continuity (two pieces, 6 by 16 in.), would give:

$$H = 154 \text{ lb. per sq. in.}$$

which, although too high, is much more reasonable than two pieces, 4 by 16 in.

In the foregoing calculations, for the sake of simplicity, the writer has not included the dead loads; and, as he has referred to longitudinal shearing, he will point out the discrepancy between the safe shearing stress with the grain, as recommended by the Association of Railway Superintendents of Bridges and Buildings and the safe uniform loads on beams as per the tables reprinted from the *Proceedings* of the same Association.

These tables are computed, as explained in the preface of the pamphlet, assuming that: "the working stress in shearing is one-twentieth of that in cross-breaking."

To realize this, take for instance the tabular figures for a short-leaf yellow pine beam, 1 in. wide, 16 in. deep, on a 12-ft. span.

The foregoing formula, $S = \frac{3}{2}\frac{V}{b\,d}$, or $V = \frac{2}{3}\,b\,d\,S$, if $S = \frac{1\,000}{20} = 50$, $b = 1$ in., $d = 16$ in., and noting that the total load is equal to twice the end reaction for uniform loads, gives:

$$W = \frac{4\,d\,s}{3} = \frac{4 \times 16 \times 50}{3} = 1\,070 \text{ lb.}$$

Therefore the tabular values correspond to a safe shearing stress of 50 lb. per sq. in., whereas the value generally accepted in ordinary practice is 100 lb. per sq. in. and therefore the safe load on the foregoing beam should be 2 140 lb. or, say, twice as much as shown in the table.

Therefore the assumption of "working shearing stress = one-twentieth of allowable extreme fiber stress" does not agree with actual practice. Doubtless this error came through overlooking these two facts:

The working cross-breaking stresses recommended by the Association of Railway Superintendents refer to the ultimate extreme fiber strain as an average between the ultimate endwise compressive and tensile strengths.

The ultimate cross-bending strains, as shown in the United States Forestry Department tables, agree fairly well with the values of R deduced from the well-known formula:

$$\text{Bending moment} = R \frac{I}{Z}$$

applied to a beam, 1 in. square, and of 1 ft. span, supported at each end, loaded with a center breaking load, W, given by actual tests.

The writer has not at hand the center breaking loads from the Forestry Department tests, but has come to this conclusion by using the values given in Trautwine's "Pocket-Book."

In the latter case it is a matter of fact that, for white pine and similar material, the ratio between shearing with the grain and cross-bending (ultimate) is about one-twentieth.

Another important point in bridge building is the depth to be given to a girder to stand a certain strain, which, as far as the writer knows, has never had a definite solution. Experience shows it to be from one-ninth to one-twelfth of the span, which is not definite. At the same time, experience shows the maximum allowable deflection of girders under good conditions, and from this the depth can be deduced as follows:

The value of the bending moment due to the actual axle loads being nearer to the value for the equivalent uniform load than to the moment due to a single concentrated load, we may refer the actual deflection of the girder to the formula for uniform loads.

For the sake of accuracy, however, the total uniformly distributed load must be computed so that the maximum bending moment it produces shall be equal to the maximum bending moment due to the actual loading diagram.

Call M the actual maximum bending moment, l the span, and W the total uniform load:

$$M = \frac{W\,l}{8}, \text{ and therefore } W = \frac{8\,M}{l} \quad \dots\dots\dots\dots (2)$$

The deflection is given by the formula:

$$f = \frac{W\,l^3}{77 \times E \times I}, \text{ and } \frac{1}{77} = \frac{5}{384}$$

From which,

$$I = \frac{W\,l^3}{77 \times E \times f} \quad \dots\dots\dots\dots (3)$$

Otherwise, call S the permissible unit stress, F the flange area, and d the depth of the girder:

$$M = S \times F \times d.$$

At the same time, assuming, as in regular practice, that the bending moment is entirely resisted by the flange area:

$$I = \frac{F \times d^2}{2}, \text{ and } F = \frac{2\,I}{d^2}, \text{ and therefore:}$$

$$M = \frac{S \times 2 \times I}{d^2} \times d = \frac{2\,S\,I}{d}, \text{ and } d = \frac{2\,S\,I}{M} \ldots\ldots\ldots\ldots (4)$$

From Equations 2 and 3,

$$I = \frac{\frac{8\,M}{l} \times l^3}{77 \times E \times f} = \frac{8\,M\,l^2}{77\,E \times f},$$ and substituting this value in Equation 4,

$$d = \frac{2\,S \times 8\,M\,l^2}{77 \times E\,f\,M} = \frac{2 \times 8 \times S\,l^2}{77 \times E \times f},$$ in which, of course, the span, l, is expressed in inches; but, if the span be measured in feet:

$$d \text{ (inches)} = \frac{2 \times 8 \times 144 \times S \times l^2 \text{ (feet)}}{77 \times E \times f \text{ (inches)}}.$$

Assuming $E = 29\,500\,000$ lb., and $S = 10\,000$ lb.,

$$d = \frac{2 \times 8 \times 144 \times 10 \times l^2 \text{ (feet)}}{77 \times 295 \times 100 \times f} = 1.014 \frac{l^2 \text{ (feet)}}{100 \times f}.$$

As for the value of f, although it might go up to the limit of 1 in. in a 100-ft. span, the writer finds, after working out the depths of the plate-girder spans on this road from 25 to 85 ft., that the results of the foregoing formula (assuming a deflection of $\frac{7}{8}$ in. in a 100-ft. span) agree with the actual dimensions within 2 or 3 in. in most cases.

The following is an example: In the 80-ft. girder span over "Los Andes" River on the Guatemala Railway, $l = 80$ ft. The deflection should be $0.875 \times 0.8 = 0.7$ in. in order to keep the standard of $\frac{7}{8} = 0.875$ in. per 100 ft. of span.

In this case

$$d = 1.014 \frac{6\,400}{100 \times 0.7} = \frac{1.014 \times 6\,400}{70} = 92.7 \text{ in.}$$

The actual depth of the girder as built by the Baltimore Bridge Company = 90 in.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

THE SIXTH AVENUE SUBWAY OF THE HUDSON AND MANHATTAN RAILROAD.

By H. G. Burrowes, M. Am. Soc. C. E.

To be Presented October 2d, 1912.

Introductory.

The Hudson and Manhattan Railroad work described in this paper is that portion built under Sixth Avenue between 12th and 33d Streets, New York City, as shown on Plate XLIX. The general plan consists of a two-track, reinforced concrete subway in the center of Sixth Avenue from the end of the iron-lined tunnels at 12th Street north to and including a three-track station at 33d Street. The extension northward from this Station to the Grand Central Station was authorized by a later franchise, and is not considered in this paper.

There are five stations. Those at 14th, 19th, 23d, and 28th Streets are similar to one another in type and practically the same in detail, with the exception of the entrances. The 33d Street Station is more comprehensive in all its details, and the structure and its general arrangements and equipment call for particular attention, it being of a new type compared with the usual stations on New York City tunnels.

In prosecuting this work, it was necessary to support temporarily the entire elevated railway, the entire surface railway, and also all

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

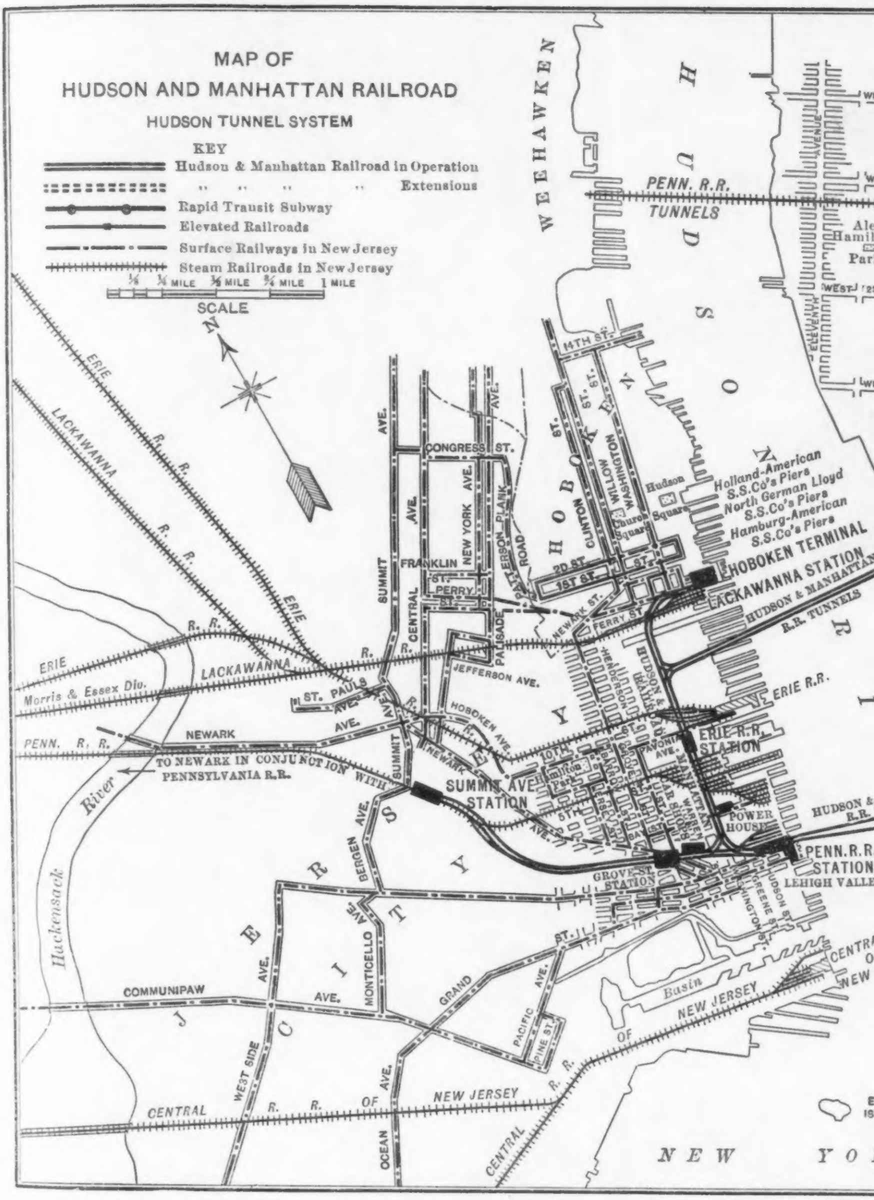
the pipes and electric cable properties between 12th and 33d Streets. Before making the excavation, it was necessary to construct temporary channels for all existing sewers and then build new sewers, one on each side of the structure; support all vaults within the limits of the work; remove and replace all vaults, and support all buildings between 31st and 32d Streets, carrying their foundations down to sub-grade; replace the elevated and surface railways on permanent concrete and brick foundations resting on the roof of the structure; relay all pipes and electric subways in the street, supporting them on timber bents; replace the electric cables in new ducts; restore the water, gas, and electric services; back-fill over the subway; and relay the asphalt pavement.

Under the conditions of the franchise, the street surface had to be maintained for the unobstructed use of traffic. This made it necessary to bridge over the entire surface and maintain the planking in first-class condition. The franchise also required that no openings be made in the street planking other than those absolutely necessary for the economic prosecution of the work.

The subway and stations throughout are of reinforced concrete, and the columns in the stations are of steel or cast iron.

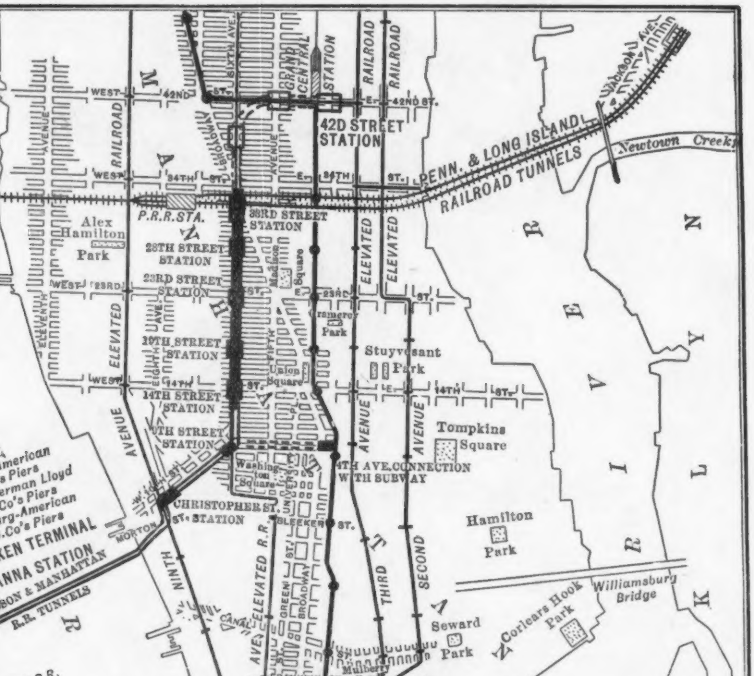
Standard Tunnel Section.—The standard tunnel section, shown on Plate LII, was used between the stations and in the work between 27th and 33d Streets, but, between 12th and 27th Streets, the walls are 4 in. thicker. At the track cross-over enlargement, between the 19th and 23d Street Stations, the section was modified by the elimination of the center wall and by changing the roof from an arch to an I-beam, concrete construction, supported on the side-walls alone. There was also a modification between 12th and 13th Streets due to the widening of each tunnel in order to provide for the reverse curve in the tracks made necessary by the difference in track centers in the standard subway and in the iron-lined tunnel below 12th Street, the distance between the tracks being 13 ft. 3 in. in the standard subways and 17 ft. 6 in. in the iron-lined tunnels.

Standard Stations.—As shown on Plates LII and LVI, the standard stations differ little in detail except in the entrances. These are built under the stairways of the elevated railway, with the exception of the east entrance of the 23d Street Station, which is through the store on the southeast corner; the east entrance of the 19th Street Station, which is through the store on the southeast corner; the west



MAP OF
HUDSON AND MANHATTAN RAILROAD
HUDSON TUNNEL SYSTEM
KEY
Hudson & Manhattan Railroad in Operation
" " " " Extensions
Rapid Transit Subway
Elevated Railroads
Surface Railways in New Jersey
Steam Railroads in New Jersey
1/8 1/4 MILE 1/2 MILE 3/4 MILE 1 MILE
SCALE
WEEHAWKEN
HOBOKEN
JERSEY CITY
HUDSON RIVER
PENN. R.R. TUNNELS
Holland-American S.S.Co's Piers
North German Lloyd S.S.Co's Piers
Hamburg-American S.S.Co's Piers
HOBOKEN TERMINAL
LACKAWANNA STATION
HUDSON & MANHATTAN R.R. TUNNELS
ERIE R.R. STATION
POWER HOUSE
PENN. R.R. STATION
LEHIGH VALLEY
GROVE ST. STATION
SUMMIT AVE STATION
TO NEWARK IN CONJUNCTION WITH PENNSYLVANIA R.R.
ERIE R.R.
LACKAWANNA R.R.
Morris & Essex Div.
PENN. R.R.
Hackensack River
CENTRAL R.R. OF NEW JERSEY
COMMUNIPAW AVE.
NEW YORK

PLATE XLIX.
PAPERS, AM. SOC. C. E.
AUGUST, 1912.
BURROWES ON
SIXTH AVENUE SUBWAY,
H. & M. R. R.



entrance of the 19th Street Station, which is through the store on the northwest corner; and the east entrance of the 14th Street Station, which is through the store on the southeast corner. Where entrances were made through stores, or private property, an agreement was entered into with the owners granting a right of way through the building. This was considered to be of advantage to the owners, as many patrons of the subway would pass through the stores, with consequent increase in their trade. At the points where such entrances were established it was necessary to make alterations in the interiors of the buildings.

The platforms of the stations rest on the foundations of the columns, on the side-walls, and on a brick wall under the overhang of the platform. At the 28th Street Station, however, a concrete girder was built, replacing the brick wall.

The roof of each station is of groined arches resting on the side-walls, the center-wall, and cast-iron columns.

Toilets are provided on each platform, and are drained by an automatic ejector operated by air power, which discharges into the sewers alongside the stations, with the exception of the 19th Street Station, where the sewer is below the level of the platform and the connection is made directly into it.

Floor drains were provided in the platforms and station entrances in the 14th and 19th Street Stations, but subsequent use has shown that they become clogged because so little water is passed through them in washing the platforms, and the dirt and sweepings remain in the drain heads. In the 23d and 28th Street Stations the platforms are sloped toward and drained to the track.

The cast-iron columns of the stations are 15 in. in diameter, 2 in. thick, and rest on bases having a bearing area of 2 304 sq. in. The area of the caps for the support of the concrete roof is 1 521 sq. in. The reinforcing rods are shown on Plate LII, and need no description.

The Thirty-Third Street Station.—This is a three-track "through" station, as shown on Plate LIX, and, for convenience in this description, will be taken to include the enlargement at the south end. Two enlargements were planned, one at each end of the station, to provide room for the track arrangements necessary to change from two tracks in the subway to three tracks in the station.

The station is built on rock foundation, and in general consists of

framed steel columns resting on concrete foundations, with side-walls and groined arch roofs and concourse floor. The roofs and the concourse floor are supported on the side-walls and on steel brackets riveted to the steel columns.

The columns of the elevated railway, excepting the two in the center of 32d Street, are supported on concrete piers based on steel grillages which span over, and distribute each column load on, four of the station columns. Each station column is in one piece, and passes through the concrete of the roof and the concourse floor. The elevated railway columns in the center of 32d Street are supported on steel towers made up of the extended columns of the station.

Over the enlargement, the elevated railway column foundations or piers rest directly on the roof, and under them extra girders are embedded in the roof for their support. These girders are supported by the side-walls and by steel columns in the center-walls.

The floor or invert of the station, between the foundations of the columns and the side-walls, is simply a concrete floor resting on the rock bed to carry the tracks, etc., and was planned to be 12 in. thick; but, owing to the unevenness of the rock surface as blasted out, it varies, sometimes reaching 3 ft.

All the station columns are 13 ft. 10 in. from center to center, and the groined arches, therefore, are in square panels and of the same radius, 8 ft. 3 in., in both directions.

The roof arches of the station are of reinforced concrete, 12 in. thick at the crown over the concourse, where the depth of fill was 10 ft., and 18 in. thick at the crown in the south end of the station, where the depth of fill varied from 18 to 20 ft.

There are five entrances to this station, as shown on Plate LIX: three under the stairways of the elevated railway station at Sixth Avenue and 33d Street, covered with metal kiosks; one through the department store between 32d and 33d Streets, and one under Greeley Park, covered with an ornamental kiosk of artificial stone. In addition to these entrances which lead directly to the street, there are three stairways leading up to the street floor of the store between 32d and 33d Streets. There are, therefore, eight entrances, all leading to the concourse floor of the station.

The station is 387 ft. long on the train platforms, providing for the unloading of 8-car trains, and, throughout its entire length,

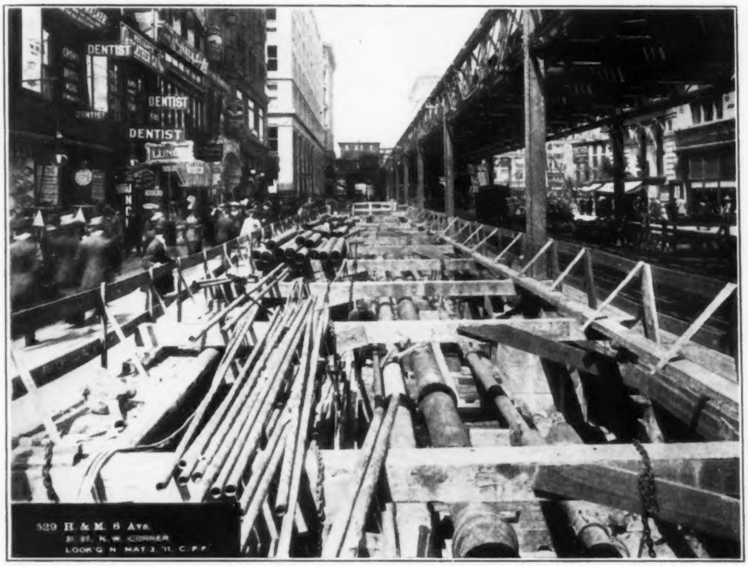


FIG. 1.—GENERAL VIEW OF STREET CONDITIONS WHILE RELAYING PIPES ON SIXTH AVENUE.

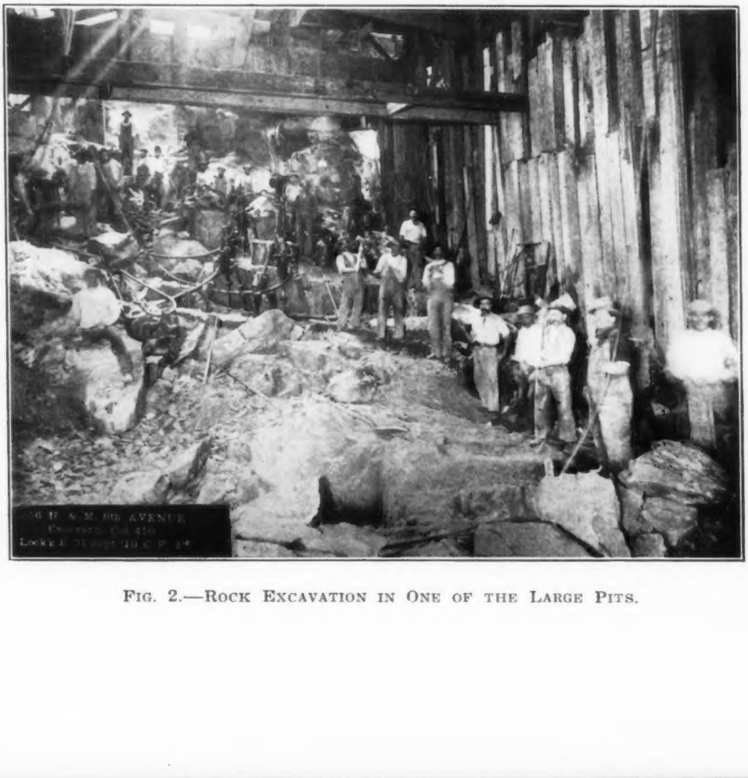


FIG. 2.—ROCK EXCAVATION IN ONE OF THE LARGE PITS.

occupies the full width of Sixth Avenue, being 94 ft. 10 in. wide between the interior faces of the side-walls.

There are two side platforms, 15 ft. $1\frac{1}{2}$ in. wide, and two center platforms, 18 ft. 5 in. wide. Over the northern half of the train platform, between 32d and 33d Streets, there is a concourse, the full width of the station and 152 ft. long. Leading to this concourse there are twelve reinforced concrete stairways, three to each of the four platforms.

The roof is of groined arches of the same type as used in the standard stations, and the walls throughout are lined with ornamental glazed tile.

The framed steel columns of the station are encased in artificial stone slabs, made on the work, and the twelve stairways between the concourse floor and the train platforms have railings and balustrades of the same material.

At the north end of the concourse there are toilets, telephone booths, and four ticket offices for the various railroads served by the tunnel trains. At the south end there are two porter's rooms and a fan-room for the exhaust fan which discharges the foul air from the subway.

A signal tower and a battery-room are at the south ends of the two center platforms for the operation of the mechanism of the cross-over, etc., of the track in the enlargement. The same provision for tower and battery-room will be made at the north end of the station when the extension is built.

Under the Sixth Avenue front of the department store between 32d and 33d Streets on the west side of Sixth Avenue, and at the level of the platform, there is a large baggage and storage-room, 194 ft. long and 69 ft. wide, with access to the west platform, for the handling of baggage. This baggage-room has a large baggage chute and an escalator for raising and lowering parcels, etc., between it and the street level at 33d Street. On the 33d Street side of the same store building there is an entrance and handling-room, at the heads of the chute and escalator, so that baggage by truck, etc., may be received from and delivered at the street level.

Over the north end of this baggage-room and at the level of the concourse floor, there is a parcel check-room which is connected with

the large baggage-room below by a small chute and dumb-waiter for small parcels.

This station has four electrically-operated fans, two at the south end of the concourse, for the discharge of foul air from the subway, and two at the south end of the large baggage-room, for the supply of fresh air to the station. The exhaust fan draws the foul air through a reinforced concrete duct laid on the roof of the station and subway from a point north of the end of the 28th Street Station, which is the high point in the grade of the east or up-town subway. The fresh-air fan supplies the air through a concrete duct laid below the foundation of the station, at right angles to the track. This duct has openings under the platforms with louvers through which fresh air is supplied. The foul air is discharged through an opening in Greeley Park; the fresh air is taken in through an opening beside the department store at 32d Street.

Two automatic ejectors, one in the middle of the station, under the west center platform, and one at the north end of the same platform, are operated by compressed air. They are set in sumps which collect the drainage of the subway and station at this point. The ejector at the north end of the platform also collects and discharges the drainage of the toilets. These ejectors lift the drainage 30 ft., and discharge it directly into the sewers.

The concourse floor is of the same type as the roofs, the thickness being 6 in. at the crown of the arches.

The platforms are of 8-in. reinforced concrete slabs resting on 13-in. reinforced girders, the ends of which are carried on the concrete jackets around the bases of the steel columns and on the side-walls. These concrete girders and slabs were built in one operation. The tops of the platform and entrance floors are finished with a 1-in. coating of granolithic pavement which was laid as soon as the concrete had set sufficiently.

Structural Steel.—At 32d Street, where the station crosses the Pennsylvania Tunnel, the track level or grade line passes through the arch of the roof of that tunnel. It was necessary, therefore, to remove the reinforced concrete arch of the tunnel and replace it with heavy girders resting on the tunnel walls. The columns of the station and the track and platforms rest on these girders.

PLATE LI.
PAPERS, AM. SOC. C. E.
AUGUST, 1912.
BURROWES ON
THE SIXTH AVENUE SUBWAY,
H. & M. R. R.

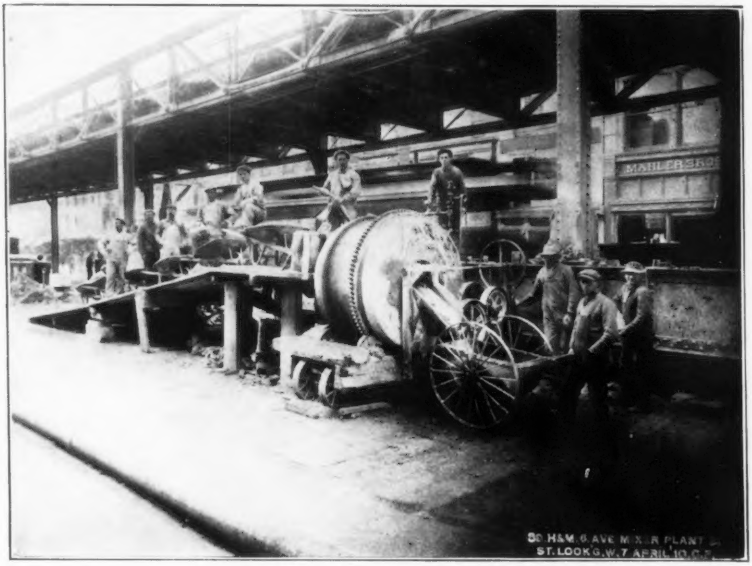


FIG. 1.—CONCRETE MIXER AND GANG.

FIG. 2.—CABLEWAY ON SIXTH AVENUE.

The steel columns of the station, as shown on Plates LII and LIX, are made up of two $\frac{5}{8}$ to $\frac{3}{4}$-in. plates and two 10-in., 30-lb. channels riveted to form box columns, and these have brackets at the springing line of the arches for their support. The bearing areas of the base castings vary from 961 to 1 296 sq. in., depending on the loads carried. The grillages on top of the station columns consist of three and sometimes four girders on which rest the four 24-in. I-beams which support the foundations of the elevated railway columns.

The twenty-four heavy girders spanning the walls of the Pennsylvania Tunnel are grouped in clusters of four under the station column lines; and the four I-beams are in two pairs near the side-walls. The six clusters of girders rest on steel billets placed between their ends. A grillage of 20-in. I-beams, embedded in the walls of the Pennsylvania Tunnel, supports the twenty-four columns of the station which are over the tunnel, a steel billet being placed under each column spanning the four girders. The columns are anchored to the billets. Eight of these station columns form the two towers supporting the two elevated railway columns in the center of 32d Street. These eight columns pass through the concrete roof of the station.

The track is carried across the Pennsylvania Tunnel on a steel trough flooring resting on angle steps fastened to the webs of the outer girders of each group. The ties of the track are laid in this trough floor, as shown on Plate LII.

Steel Reinforcing Rods.—In each panel of the roof over the concourse there are two sets or grids of rods laid so that the nearest rod is 2 in. below the top face of the concrete and 2 in. above the crown of the arch. The upper set of rods in each panel is composed of seven 1-in. square rods laid each way, transversely and longitudinally. The lower set is composed of fourteen 1-in. square rods laid each way in each panel. In each panel of the roof over the south end of the station, where the load is greater, the upper set of rods is 6 in. below the top surface of the concrete and consists of seven 1-in. square longitudinal and fourteen 1-in. square transverse rods per panel. The lower set is 2 in. above the crown of the arch and of the same number per panel both ways as in the upper set. In the concourse floor there is only one set of rods, and these are 2 in. above the crown of the arch, there being fourteen transverse and seven longitudinal rods per panel, and all are 1 in. square. Around the steel columns (where

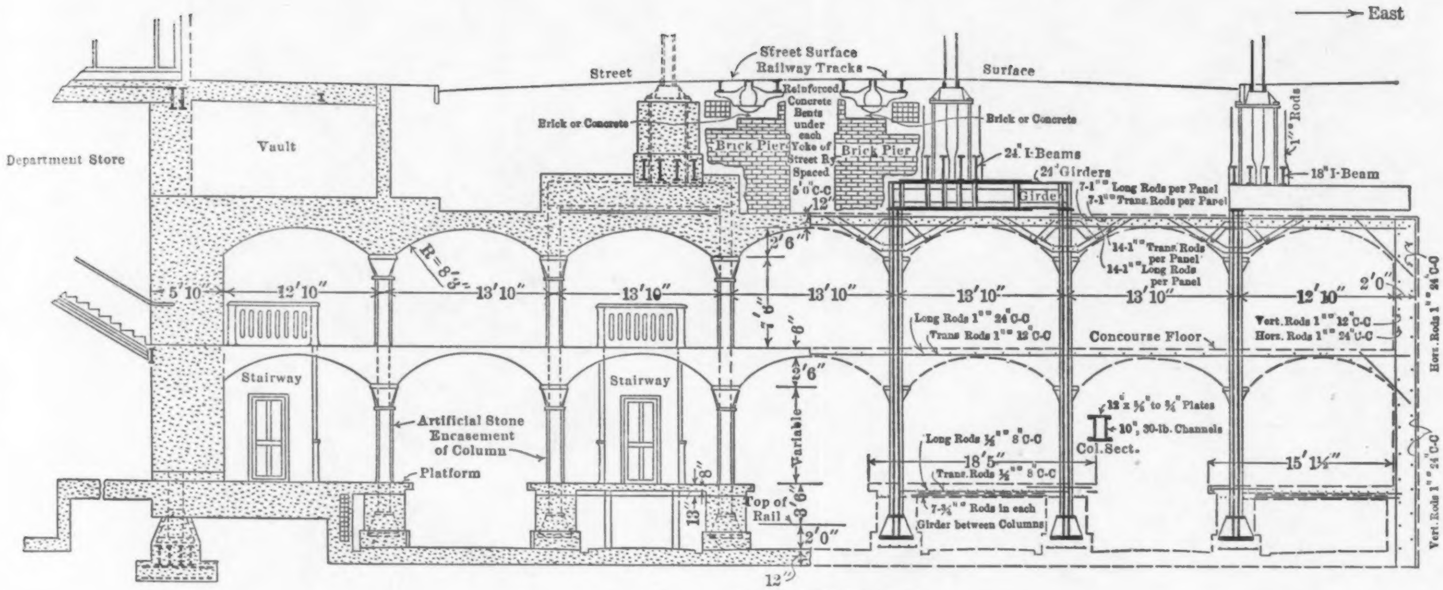
they pass through the roof) there are additional special bent rods as shown on Plate LII, in the form of a basket cluster. The reinforcing rods in the side-walls are 1 in. square, and 2 in. from the faces of the walls. The vertical inside rods are 12 in. from center to center and the horizontal rods 24 in. from center to center. The vertical outside rods are 9 in. from center to center and the horizontal rods 24 in. from center to center. The rods in the platform girders and slabs are 2 in. from the lower sides. There are seven $\frac{3}{4}$-in. square rods in the bottoms of each girder and $\frac{1}{2}$-in. square rods 8 in. apart each way in each slab or panel formed between the columns of the station.

There are 1-in. square rods 2 in. from the outside faces of the 5 by 5-ft. elevated railway column piers, and 11 in. from center to center, to reinforce the piers, because it was impossible to increase the areas of the bottoms of the piers, owing to the position of the steel grillages supporting them. This reinforcement in the column piers was placed where the depth exceeded 6 ft.

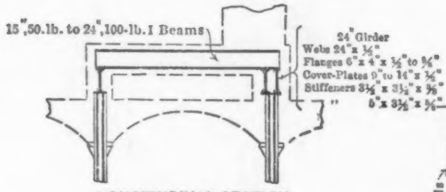
Station Finish and Decoration.—After the completion of the station walls, floors, roofs, and platforms, the interiors were finished and decorated. The floors and platforms are finished with a granolithic top. The side-walls are tiled throughout, from the floors to the springing lines of the roof arches, with 6 by 6-in. white tile with green trim, laid in a mortar bed. All the stairways are equipped with wooden hand-rails, and the steps with iron safety treads embedded in the granolithic top finish. The ceilings and the faces of the center walls are finished with a wash of white cement and white sand or marble dust, applied with a brush. In the stations at 14th, 19th, and 23d Streets, the ceilings were first plastered to give a smooth appearance, but this was not done in the 28th and 33d Street Stations because it was decided that the form marks, or visible lines left by the lagging, were not detrimental to the appearance, as they conformed to the lines of the arches.

The iron railings at the ticket booths and the cast-iron columns are painted with an enamel green, conforming to the station color. The woodwork of the ticket booths, etc., is painted or varnished to conform to the color of the station.

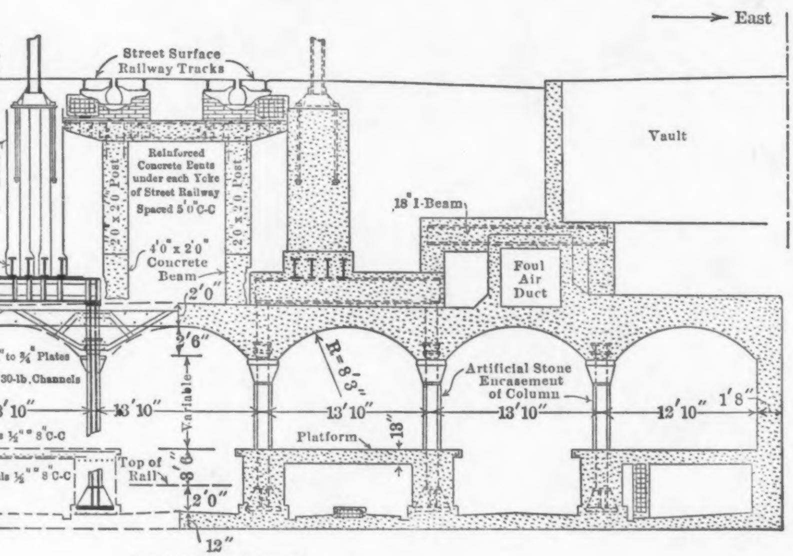
The steel columns of the 33d Street Station are encased in moulded slabs made of white cement and marble dust, and the railings and



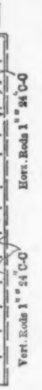
TRANSVERSE SECTION
North Half of 33d Street Station

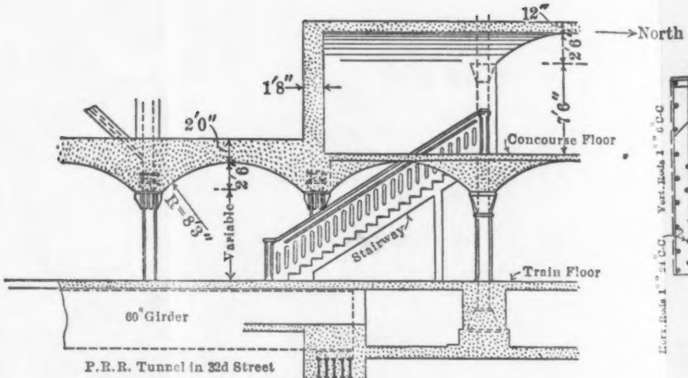


LONGITUDINAL SECTION
Girder-I-Beam Grillages for Elevated Railroad Columns

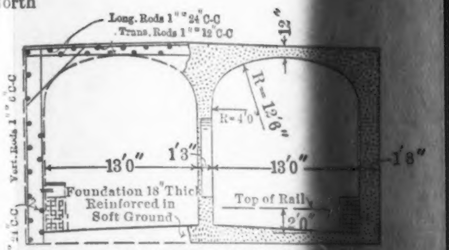


TRANSVERSE SECTION
South Half of 33d Street Station

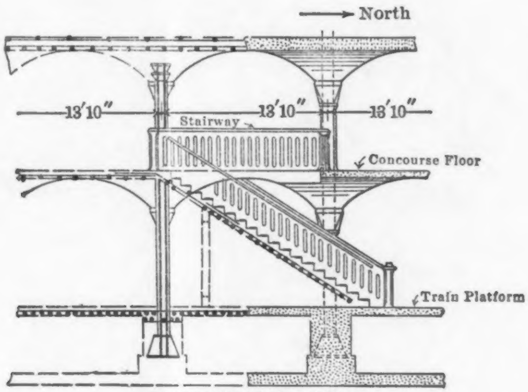




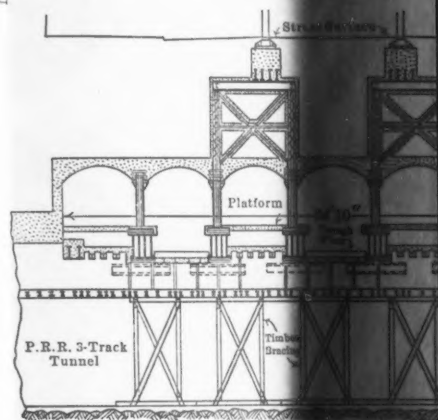
LONGITUDINAL SECTION
AT END OF CONCOURSE.



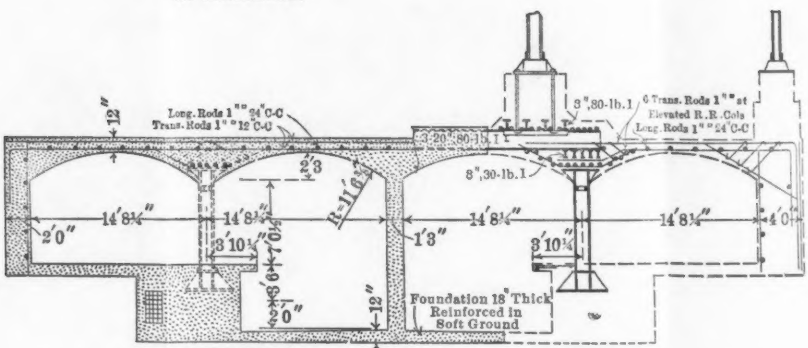
STANDARD TUNNEL SECTION
BETWEEN STATIONS.



LONGITUDINAL SECTION
AT CONCOURSE.

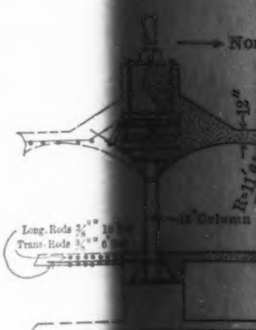


TRANSVERSE SECTION AT 32D S

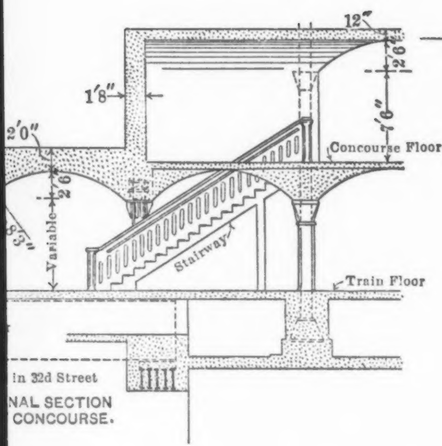


HALF SECTION
Standard Station-between Columns.

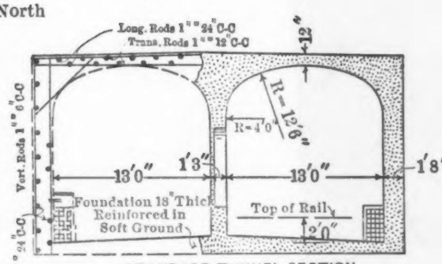
HALF SECTION
Standard Station-at Elevated Railroad Column.



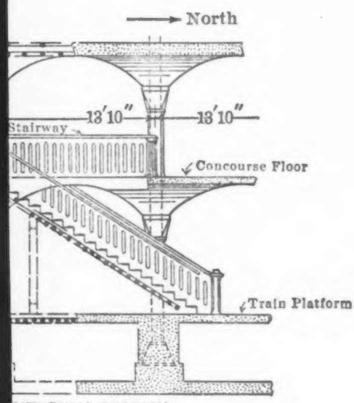
LONGITUDINAL SECTION



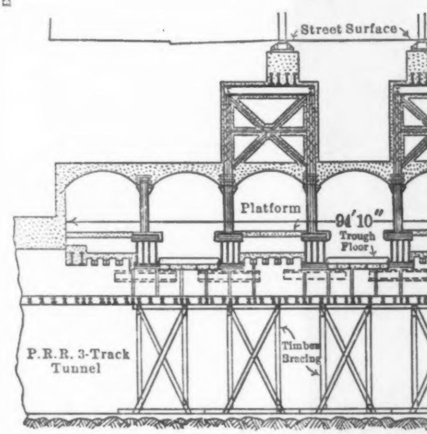
NAL SECTION
CONCOURSE.



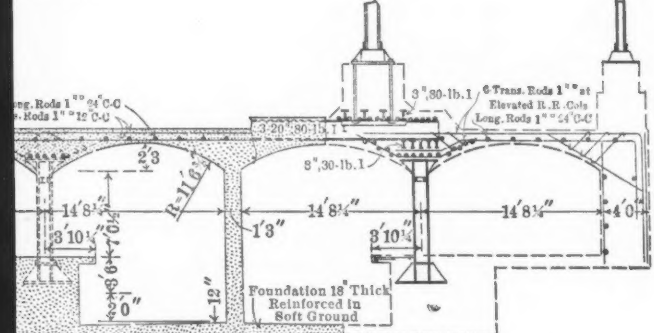
STANDARD TUNNEL SECTION
BETWEEN STATIONS.



GITUDINAL SECTION
AT CONCOURSE.

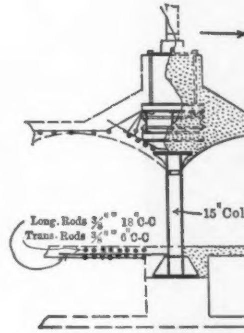


TRANSVERSE SECTION AT S



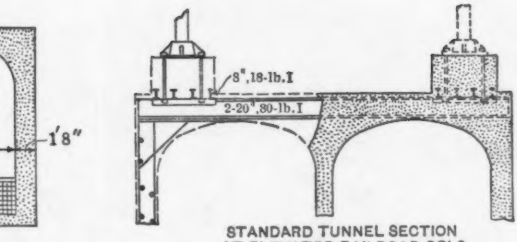
HALF SECTION
tion-between Columns.

HALF SECTION
Standard Station-at Elevated Railroad Column.

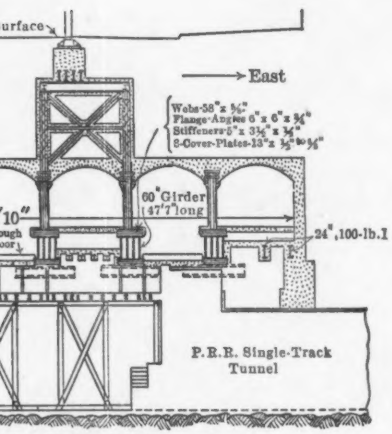


LONGITUDINAL SE

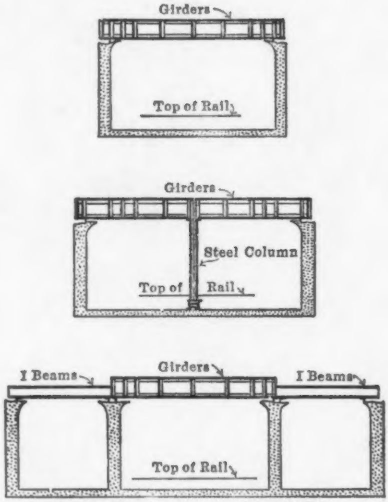
PLATE LII.
PAPERS, AM. SOC. C. E.
AUGUST, 1912.
BURROWES ON
SIXTH AVENUE SUBWAY,
H. & M. R. R.



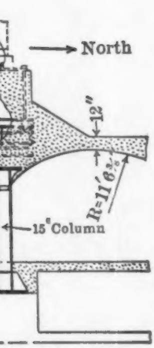
STANDARD TUNNEL SECTION
AT ELEVATED RAILROAD COLS.



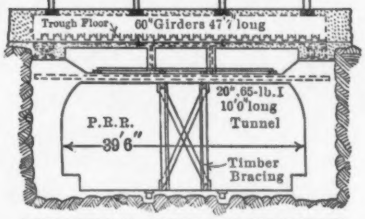
TION AT 32D STREET.



SECTIONS OF ENLARGEMENTS.



DINAL SECTION.



LONGITUDINAL SECTION AT 32D STREET.

SECTIONS OF TUNNELS
STATION & ENLARGEMENTS.

balustrades of the stairways between the train and concourse floor, are of the same material. All this artificial stone was made on the work.

Enlargement.—The enlargement south of the 33d Street Station is on rock foundation, and in general consists of one or two center-walls, excepting at the track cross-over, and two side-walls, all resting on rock. These walls support a flat roof made up of steel girders embedded in concrete on which rest the elevated railway, the surface railway, the street, etc. This enlargement is 280 ft. long, between the standard tunnel and the 33d Street Station, 27 ft. 3 in. wide at the south end and 92 ft. 4 in. wide at the north end. The thickness of the side-walls is 18 in. at the south end and 24 in. at the north end. The center wall is 15 in. thick, embedding the steel columns supporting the roof girders.

Street Surface Car Tracks.—The surface car tracks, shown on Plates LII, LXII, and LXV, are supported by brick piers and reinforced concrete bents under each yoke of the car track, 5 ft. from center to center. Over the concourse, and where the depth between the yokes of the street car tracks and the top of the roof is less than 7 ft., the piers are of brick and rest on the roof of the tunnel and stations. These piers are built transversely across the street car tracks. Over the south half of the 33d Street Station and over the enlargement and tunnel, where the depth between the car track yokes and the top of roof is from 7 to 17 ft., there are reinforced concrete bents. Over the south half of the 33d Street Station, two plain concrete girders, 24 in. wide and 4 ft. deep, are set longitudinally in order to distribute the loads of the reinforced concrete bents. These bents were cheaper than brick piers for depths exceeding 7 ft. The legs of these bents rest on the plain concrete girder built on the roof.

Buildings.—The architects of the store building on the west side of Sixth Avenue between 32d and 33d Streets, having the construction of the subway in mind, carried the column foundations of that building down to the sub-grade of the tunnel, so that it was not necessary to underpin this building. Between 31st and 32d Streets on both sides of Sixth Avenue it was necessary to support the buildings temporarily and carry their foundations down to rock or to sub-grade.

Drainage.—In addition to the two sumps in the 33d Street Station, there are two others to provide for the drainage of the subways.

These are equipped with automatic ejectors which discharge into the sewers.

Air Line.—An air line in each subway supplies compressed air for operating the signals, pumps, or ejectors, and for the use of air tools and in making necessary track repairs. These air lines are of wrought-iron pipe, and are reduced in diameter from 8 in. at 12th Street to 3 in. at 33d Street.

Subway Ducts.—There is a duct bank in each tube between 12th Street and the 33d Street Station. There are splicing chambers at each end of each station in both tunnels, and also between stations, where necessary to shorten the length of cable haul.

Water-Proofing.—The subway and stations between 12th and 27th Streets are water-proofed with the regular burlap and pitch, and between 27th and 33d Streets with a compound or powder mixed with the concrete.

General Methods.

From 12th to 27th Streets.—The first step, as shown on Plate LXII, was to excavate pits between the columns of the elevated railway to or near sub-grade, and, in them, build up the timber bents to carry the girders which supported the elevated railway columns.

Between 12th and 18th Streets (excepting at 14th Street and because of wet foundation), pits were dug to within 8 or 12 ft. of sub-grade, between the elevated railway bents to be supported in the first operation, then timber piles were driven in the bottoms to hardpan or rock, and timber bents were built and framed on the piles to support the girders. These bents were carried above the street surface, and carried the I-beams or girders supporting the elevated railway columns.

Between 18th and 20th Streets, the pits between the elevated railway bents were carried down to sub-grade, the subsoil being dry. Between 20th and 27th Streets, the pits were carried down to rock, and on it the timber bents were built.

After the elevated railway columns were underpinned in the first step, the cores of excavation, between the pits and under the columns, were removed, as shown at *A*, and the piece or length of subway or station under each column supported was built. The column was then set on its final foundation on the roof of the subway or station, as shown at *B*. Then the timber bents between *A* and *B* were removed,

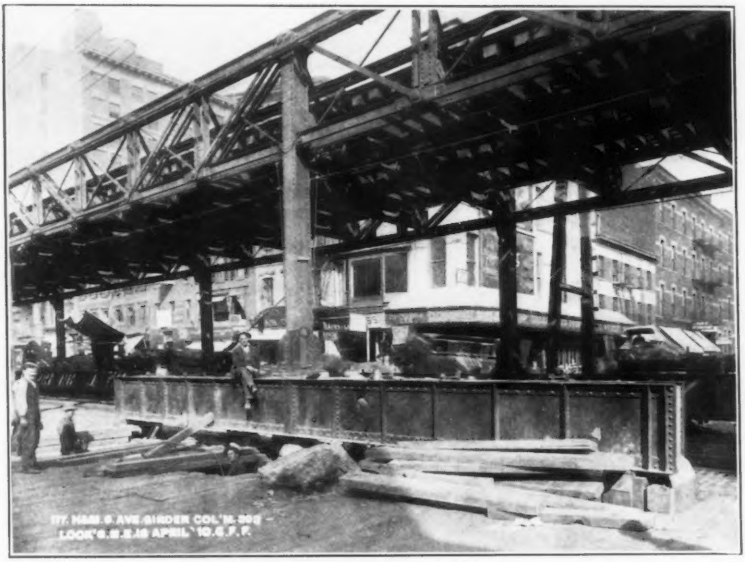


FIG. 1.—COLUMNS OF ELEVATED RAILROAD SUPPORTED ON 36-INCH GIRDERS.

FIG. 2.—ELEVATED RAILROAD SUPPORTED BY OVERHEAD TRUSSES.

and the closure pieces of subway or station between *C* and *D*, were put in, connecting the pieces of subway or station built under the elevated railway columns. The condition after this step was that four of the elevated railway bents, or eight columns, in any one block rested on their final foundations on the roof, and the only remaining columns were those at the street intersections and in the middle of the blocks, which were still on their original brick foundations. The next step, therefore, was to support these remaining two bents or four columns on the other legs of the timber bents already built in the first operation, and remove the cores of excavation under the columns. The pieces of tunnel or station under these columns were then built and the columns were set on their final foundations on the subway roof. The girders and timber bents which last supported these columns were removed and then the closure pieces of subway or station were built. Meanwhile, during the completion of the closure pieces of subway or station, the brick piers for the final foundation of the surface car tracks were built on the roof under each yoke of the surface track structure. The duct benches of the track structure were built on a dry rock packing wall laid up between the brick piers and resting on the roof of the tunnel or station, as shown at *D*.

The sewers were built as the work progressed, and the spaces behind the walls and over the subway or stations were back-filled and puddled to within 7 ft. of the street surface.

Timber bents were built every 6 ft. on each side of the subway on firm foundations of earth or rock, as the case might be, to support the gas and water pipes and electric duct lines and prevent settlement with the fill.

The gas and water pipes and electric duct lines were then relaid, and the street was back-filled and puddled, up to the surface. A temporary block pavement was laid on the fill at the level of the street surface; this was left in place until the fill had settled sufficiently, and then the asphalt pavement was put down, completing the work.

From 27th to 31st Streets.—The methods and progressive steps in this section were similar to those in the blocks between 12th and 27th Streets, the only essential difference being in the first operation, which was to underpin temporarily the columns of the elevated railway, and, after the removal of the earth excavation, to build

temporary concrete foundations under these columns from the top of the rock, so that they rested thereon and were in safe condition for the excavation of the earth and rock under those adjoining. The foundations of the columns in the center of the cross streets and in the middle of the blocks were carried down to rock temporarily, and then the succeeding operations were similar to those in the work between 12th and 27th Streets.

From 31st to 33d Streets, and the 33d Street Station.—The methods in this section, as shown on Plate LXV, differed from those in the work between 12th and 31st Streets, owing to the following conditions: the station occupied the full width of Sixth Avenue, 100 ft.; the station roof was to be of the groined-arch type; and the positions of the elevated railway columns were eccentric with reference to the station columns. For these reasons, it was necessary to build larger pieces of station under the columns in order to provide a safe foundation for them while excavating the adjoining cores. It was found necessary to support the elevated railway columns in such a manner that there would be sufficient space between the temporary column supports in which to build these larger pieces of station under the columns.

The first step was to underpin temporarily the alternate columns and carry their foundations down to rock by temporary foundations of concrete. This, of course, made these columns safe while the intermediate cores of rock were being excavated. Large overhead trusses, resting on the temporary concrete foundations of the alternate columns, were then erected to support the intermediate columns. Then light steel trusses were placed under the street surface planking, spanning the space required for the section of station to be built, and the street surface and all pipes and other properties in the street were supported on them. This left the entire space between the bearings of the light trusses free from encumbrances such as timber or other supports of the street. The rock under the columns supported on trusses was then excavated, and sections of station were built up under them. Then these columns were placed on their final foundations, supported by the steel columns of the station. After this operation, the trusses were removed and the columns, which had been first underpinned temporarily, were again underpinned and the temporary foundations and rock cores were removed from under them. The closing sections

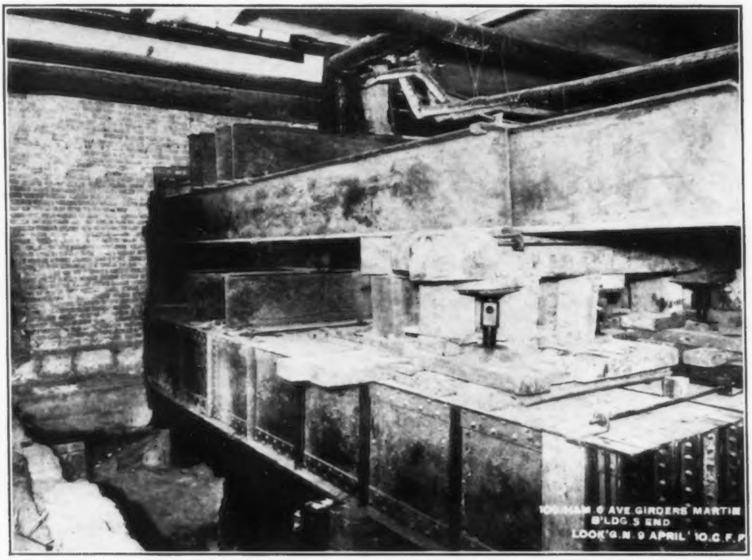


FIG. 1.—STEEL UNDERPINNING OF A COLUMN OF THE MARTIN BUILDING.

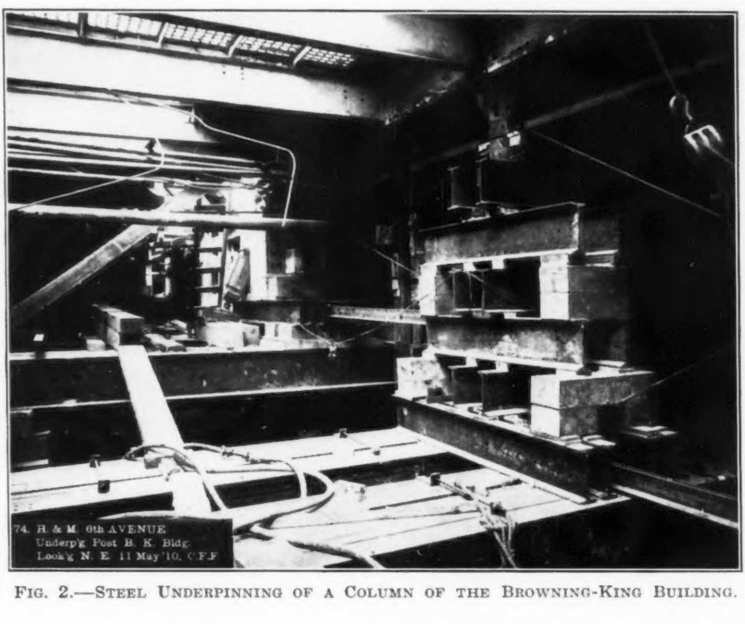


FIG. 2.—STEEL UNDERPINNING OF A COLUMN OF THE BROWNING-KING BUILDING.

or lengths of the station were built under these remaining columns and they were set on their final foundations carried on the steel grillages supported by the steel columns of the tunnel station, thus completing the gaps between the pieces of station first built.

During these operations, the buildings on both sides of Sixth Avenue between 31st and 32d Streets were underpinned. Those on the west side (from four to six stories) were underpinned in the usual manner, and their foundations were carried down to solid rock. The underpinning of the buildings on the east side, was a different matter, as the slope of the rock required that it be removed from under the faces of the buildings and to a distance of from 15 to 20 ft. inside the building line. To insure safety, it was necessary to excavate all the rock to the stratum the toe of which cleared the inside lines of the new foundation walls of the buildings at the tunnel sub-grade.

In underpinning these buildings, the rock excavation for the subway station was carried east as far as the point at which the top of the natural slope of the strata safely cleared the original foundations of the buildings. The columns and walls of the buildings were then underpinned, as shown on Plate LXV, and the rock prism was removed so that the natural slope of the rock cleared the line of the proposed wall. Then the foundation was carried down to sub-grade and the station at that place was completed. The work in the interior of the station was then prosecuted, and the track and other details were completed.

The Crossing of the Pennsylvania Tunnel at 32d Street.—The grade line of the subway passed 5 ft. 6 in. below the top of the roof arch of the Pennsylvania Tunnel. This required that this arch, 5 ft. thick at the crown, be removed and a steel girder construction be built, as shown on Plate LII. The elevated railway columns resting on the roof arch of the Pennsylvania Tunnel were supported temporarily and the tunnel arch was removed.

The reinforced concrete arch of the Pennsylvania Tunnel was drilled and blasted away, down to a point 10 ft. above the springing line of the arch, or 2½ ft. above the mezzanine floor, and the girder crossing was built. The columns resting on the girder crossing passed through the station roof and formed the towers on which were supported the foundations of the two elevated railway columns in 32d

Street. The four platforms of the station were supported by these girders and I-beams.

Underpinning the Elevated Railway.

From 12th to 31st Streets.—It will be readily seen that the existence of the Sixth Avenue Elevated Railway structure over the proposed location of the tunnels was the controlling factor in determining the method of prosecuting the work. With the exception of about ten of the lighter columns (those supporting the station platforms beyond the lines of the excavation), the entire elevated railway structure was supported temporarily and placed on new foundations on the roof or walls of the subway.

It was specified by the Interborough Rapid Transit Company that the maximum number of main structure columns removed from their original foundations at any time and supported temporarily should be four bents of two column each, out of the six bents in any block of Sixth Avenue. Between 12th and 31st Streets it was decided to adopt a system of underpinning four bents and leaving two bents on their original foundations for the anchorage of the structure while carrying on the work. In any block, therefore, the usual plan was to leave the columns at the street intersections and in the middle of the blocks on their original foundations and to underpin the other four bents, and, after setting these four bents on their final foundations on the roof of the tunnel or stations, to underpin the first two bents and set the four columns on their final foundations. At some points, such as at 14th, 18th, 23d, and 28th Streets, this system was modified, but, in general, it was followed.

The loading on each elevated railway column was 45 tons on those between the stations and from 45 to 80 tons on those at the stations.

It will be readily seen that, as the elevated railway is a pin-connected structure, it was possible to support it only at the columns.

To support these columns, brackets, with the same spacing of rivet holes as the columns, were made, having from twelve to twenty-four 1-in. holes in each. Rivets were knocked out of the columns of the elevated railway structure at the places where brackets were to be fastened, and the brackets were riveted to the columns through the rivet holes of the columns. The number of rivets in the brackets and the sizes of the brackets depended on the loading of the columns. The

FIG. 1.—STEEL PLUGS AND LATERAL BRACING OF COLUMNS USED IN UNDERPINNING THE BROWNING-KING BUILDING.

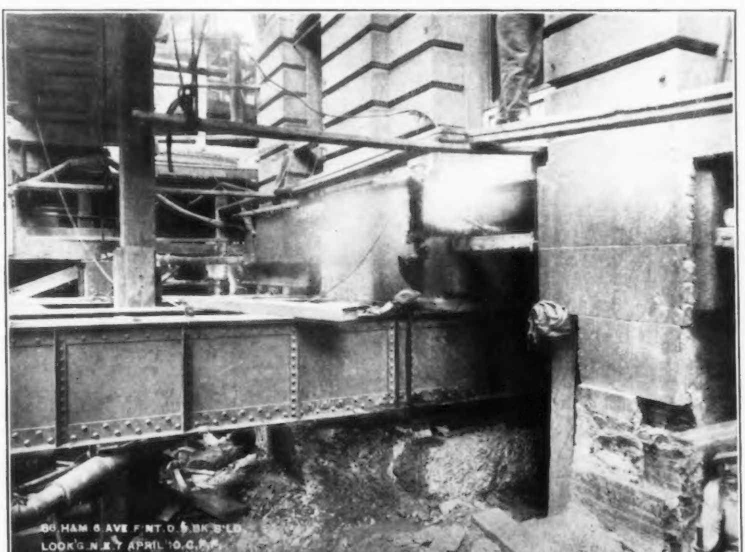


FIG. 2.—GIRDER NEEDLES USED IN SUPPORTING THE BANK BUILDING.

underpinning or framing for the temporary support of each column of the elevated railway was made up of two timber bents supporting the two 24-in. I-beams, 26 ft. long (or 36-in. girders, 39 ft. long), on which rested the column brackets already described. These timber bents were made up of two 12 by 12-in. legs, a 12 by 12-in. cap, and a 12 by 12-in. sill which rested on the concrete floor or on piles in the bottom of the pits dug to receive the underpinning timbers. When wet soil was encountered in the pits at or near the sub-grade of the tunnel, timber piles were driven in the pits down to hardpan or rock, and the timber bents were framed on them. Four timber piles were driven for each timber bent, or eight for each elevated railway column. These piles were driven with an ordinary steam pile-driver running in the leads of a common pile-driver frame provided with drop leads to allow the steam hammer to be lowered below the surface of the street.

Between 17th and 18th Streets, which was one of the localities where the foundation was wet, the station of the elevated railway prevented the use of the pile-driver, because there was not clearance enough for the frame, and here pipe piles were driven with the aid of jacks and a water jet. These pipe piles were driven in the following manner: Two brackets were riveted to the elevated railway columns to be supported, and two 24-in. I-beams were placed with their ends under the brackets. At the other ends of the I-beams, pits were dug to receive the pig-iron counterweights, which were slung to the ends of the beams. A 60-ton hydraulic jack was then set under the beams and on top of the pipe to be driven. The pipe was then jacked down, using the two beams as a counterweight against the resistance of the pile. The columns, if necessary, would have afforded a weight of 60 tons, at one end of the beam, and the weight of pig iron, of course, could be adjusted to meet any weight equal to it. The pipes were driven in small lengths, which were screwed on, one after the other, as each pipe was driven.

As shown on Plates LXII and LXV, the timber bents were braced transversely and longitudinally with 6 by 12-in. timbers, and steel knee-braces were placed between the timber legs and the bottoms of the beams or girders. These braces were made up of two 4 by 3 by $\frac{3}{8}$-in. angles. Between the brackets on the columns and the beams or girders, a 2-in. oak block and two 2 by 24-in. oak wedges were inserted, by which the column was raised in cases of small settlement.

The columns of the elevated railway were maintained at all times within $1\frac{1}{2}$ in. of their original elevations.

At the commencement of the work each column of the elevated railway was supported on two 24-in. 100-lb. I-beams, 26 ft. long, which rested on the timber bents, but this required the digging of two pits between the columns, or two pits for each column. This process was so slow, so expensive in excavation, and reduced so much the sizes of the pieces or lengths of subway to be built under each column in the first operation, that the span of the underpinning beams was increased by substituting for the I-beams two 36-in. girders, 39 ft. long.

It is evident that this improvement in framing the temporary underpinning greatly facilitated the work of building the subway, and effected a large saving in time and money. The digging of one pit 14 ft. wide, north and south, for each column, instead of two pits 8 ft. wide per column, greatly reduced the cost of the excavation of the subway, as it was practically as cheap to dig the 14-ft. pit and erect the timber tower in it as it was to dig the 8-ft. pit and erect the single bent in it with the framing. This was especially noticeable in wet ground.

When the 39-ft. girders were adopted, four timber bents were placed under the ends of the girders, 6 ft. from center to center, and framed together in the form of towers in the pits. This massing of the timber bents and framing removed much of the obstruction to the concrete work of the subway.

At the stations of the elevated railway, the columns had greater loadings. Where there were the heaviest loads, 42-in. girders were used, with the same spans as the 36-in. girders, or the span of the latter was decreased. The 36-in. girders were made up of $\frac{5}{16}$-in. webs, four 6 by 4 by $\frac{9}{16}$-in. base angles and thirty-two 5 by 3 by $\frac{5}{16}$-in. and $\frac{7}{16}$-in. angle braces or stiffeners. The 42-in. girders were made up of $\frac{3}{8}$-in. webs, four 6 by 4 by $\frac{5}{8}$-in. base angles and thirty-two 3 by 3 by $\frac{5}{16}$-in. and $\frac{7}{16}$-in. angle braces or stiffeners.

From 31st to 33d Streets.—After every alternate column of the elevated railway had been temporarily underpinned, a temporary concrete foundation was built up under it from the rock, so that the columns would be carried safely on the rock while the excavation was being done under the intermediate ones. These temporary concrete

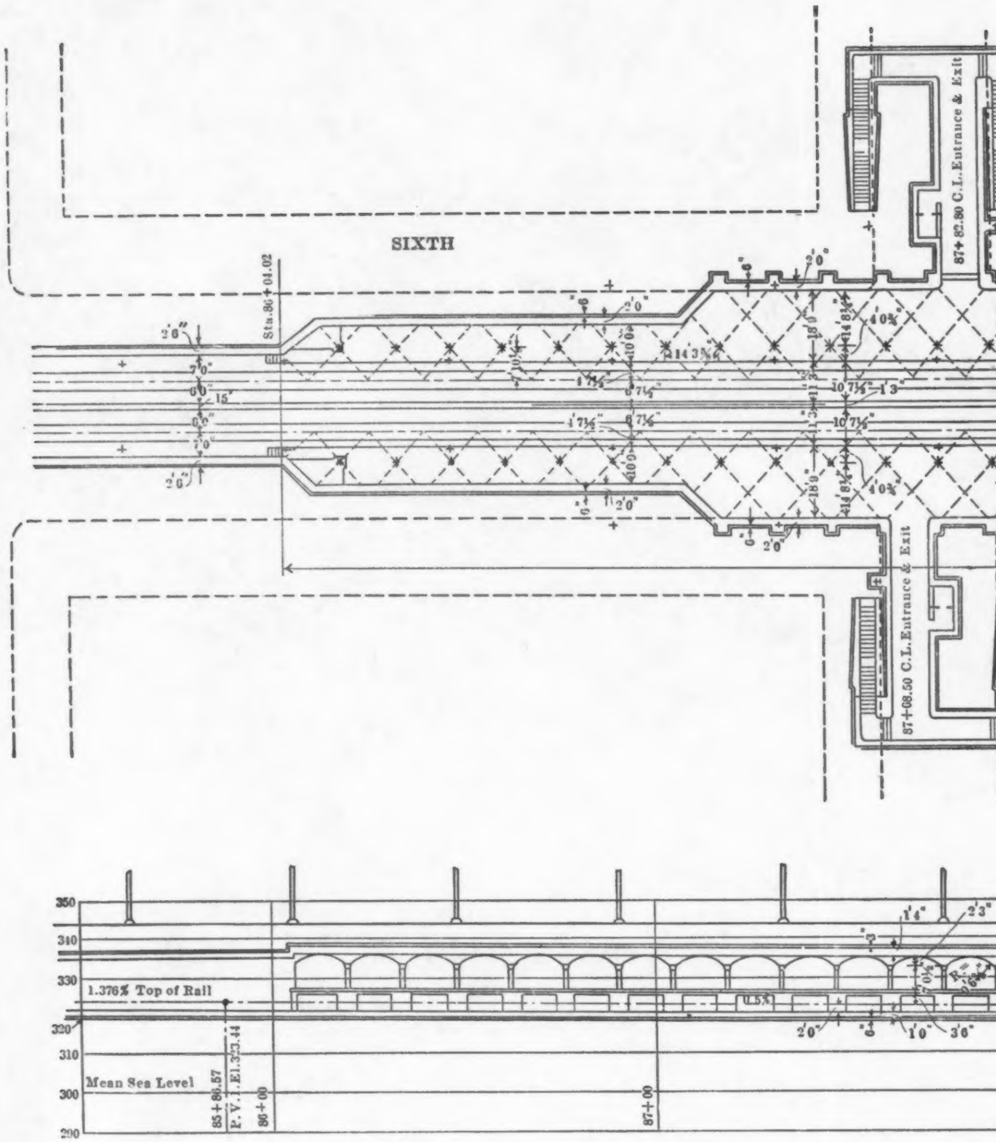
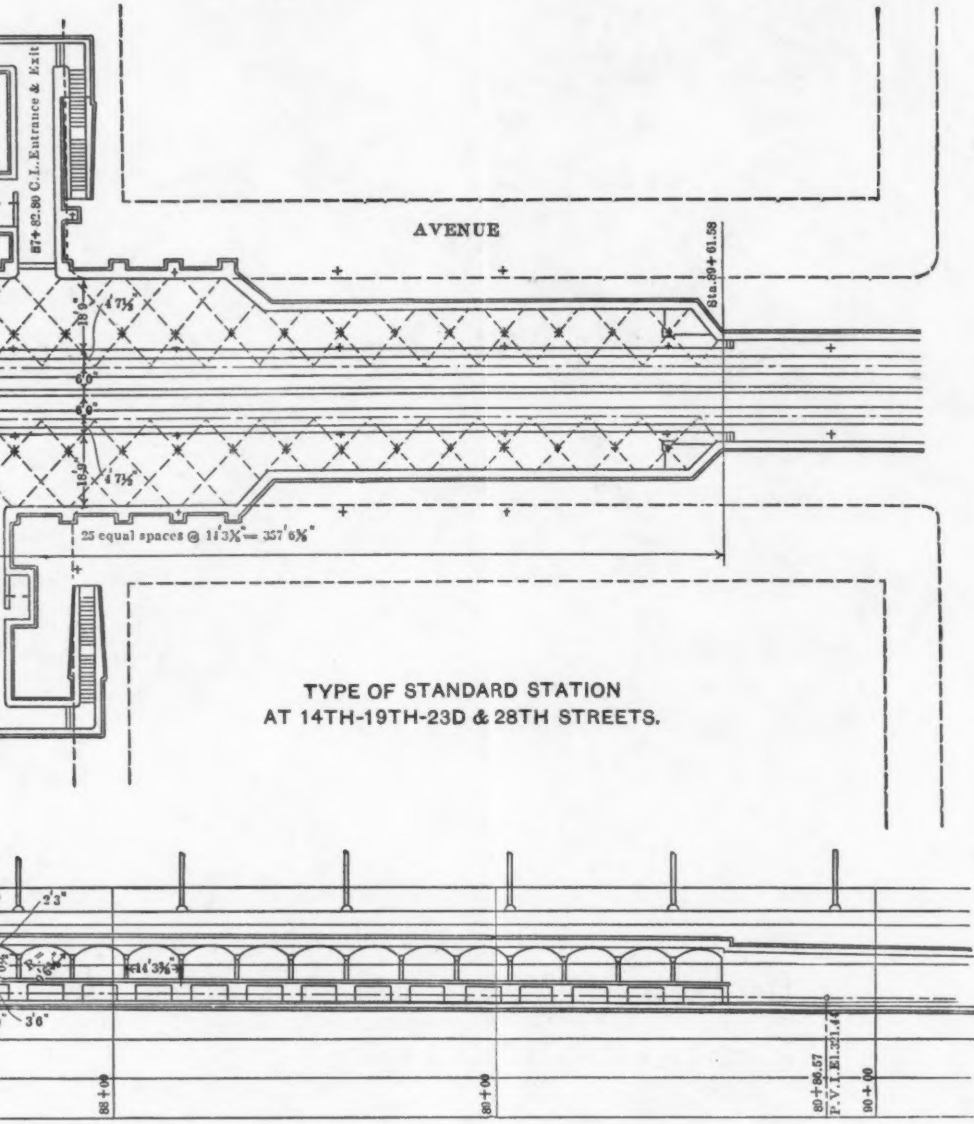

SIXTH
Stn.86+04.02
87+82.80 C.L. Entrance & Exit
87+68.50 C.L. Entrance & Exit
1.376% Top of Rail
Mean Sea Level
85+86.57
P.V.I. El.323.44
86+00
87+00
350
340
330
320
310
300
290

PLATE LVI.
PAPERS, AM. SOC. C. E.
AUGUST, 1912.
BURROWES ON
SIXTH AVENUE SUBWAY,
H. & M. R. R.



TYPE OF STANDARD STATION
AT 14TH-19TH-23D & 28TH STREETS.

foundations were made large enough to support the ends of the large trusses, as shown on Plate LXV. The steel trusses were then erected on the temporary foundations, and had a span of 75½ ft. The top chords of the trusses were bolted to the elevated railway columns, through the rivet holes of the columns (from which the rivets had been removed), and a bracket was riveted to the outside faces of the columns above the outside girders of the trusses. An oak block and oak wedges were inserted between the outside girders and the bracket and between the inside girders and the transverse floor-beams of the elevated railway structure at the columns, to give additional bearing on the trusses. Each truss at each column was made up of two girders, 62 ft. long, supported by two box posts, or legs, framed into the ends of the girders with a bottom chord composed of two 8-in. channels attached to the bottoms of the legs by a pin connection. The pins connected the legs and a cast shoe which rested on a grillage of three 8-in. I-beams, the latter being embedded in the temporary concrete foundations. The girders were made up of ⅜-in. webs, four 6 by 6 by ¾-in. base angles, sixteen 6 by 6 by ¾-in. angle braces, or stiffeners, and six 1-in. cover-plates, three at the top and three at the bottom. The posts or legs were made up of two ¾-in. plates and two 15-in., 35-lb. channels.

On the completion of the lengths of station under the elevated railway columns which were supported by the trusses, the columns were set on their final foundations, supported by the steel columns of the tunnel station, and then the trusses were removed. The elevated railway columns which were on temporary concrete foundations were again underpinned on girders in the same manner as used in the general underpinning between 12th and 31st Streets.

The columns of the elevated railway at the curb lines, which were not main structure columns, were underpinned either in the same manner as between 12th and 31st Streets or were supported on the street surface itself, where it was simply necessary to carry their foundations down to the rock or to the sub-grade of the subway.

Underpinning Buildings.

All the buildings on Sixth Avenue between 31st and 32d Streets, except those on the northeast and northwest corners of 31st Street, which are only 20 ft. high, were supported temporarily while their

foundations were being carried down to rock or to sub-grade. Before the work was started, arrangements were made with the tenants for underpinning and restoring the buildings. On the west side the buildings were of brick, from four to six stories in height. They were underpinned in the usual manner, and before the excavation was well under way in this block, 24-in. I-beams being set under the walls and supported on timber blocking inside the buildings and outside in the excavation for the tunnel. A trench was then excavated to the rock and a 3-ft. concrete wall was built from 2 to 3 ft. below the original foundations, and with its outside face at the building line of Sixth Avenue. The brick walls of the buildings were built on this concrete wall and then the I-beams were removed. The excavation of the rock below the bottom of the concrete wall was done in the usual manner by "line-holing" the rock to make a straight and even break, as the station extended the full width of Sixth Avenue and to within 7 in. of each building. The reason for carrying the foundations of the buildings on the west side of Sixth Avenue to the rock only, and not to sub-grade, was the fact that the strata dipped northeast, and thus aided in making the excavation of rock in front of the building safe.

The four buildings underpinned on the east side were: A 20-ft. front, four-story brick, between two large buildings; a 59-ft. front, eleven-story, steel frame building, designated as the "Martin Building," which was shored at all points, with the exception of the south wall; a 59-ft. front eight-story steel frame building, with cast-iron columns, designated as the "Browning-King Building"; and a 40-ft. front, five-story granite and marble-faced building without steel framing, designated as the "Bank Building." These buildings are shown on Plate LXV.

The rock was removed as close to the existing foundations as safety allowed, the buildings were shored up, the rock was removed from under their fronts, and then their foundations were carried down to sub-grade.

The Martin Building.—The front of this building was carried on four framed steel columns, the two end ones being embedded in the side-walls. It was necessary to shore up the north wall and column and the two center columns. The south wall and column rested on rock, and, being at a safe distance from the excavation, required no

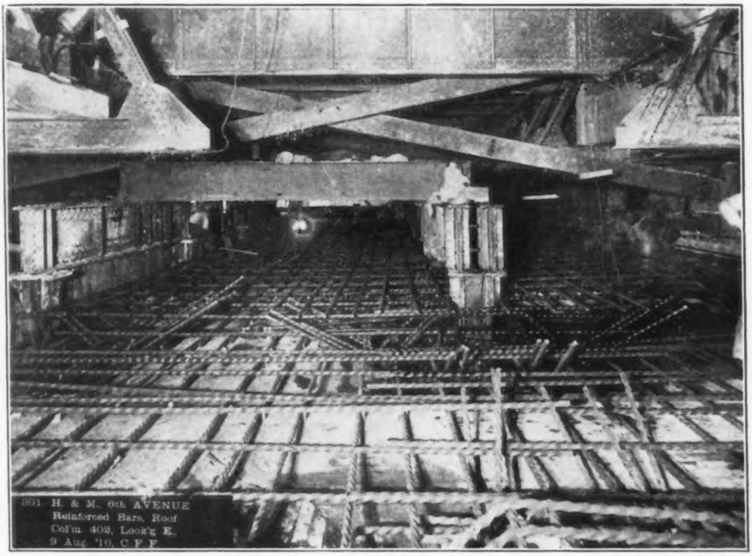


FIG. 1.—REINFORCING RODS, GROINED ARCH FORMS, AND STEEL GRILLAGES FOR FOUNDATIONS OF ELEVATED RAILROAD COLUMNS.

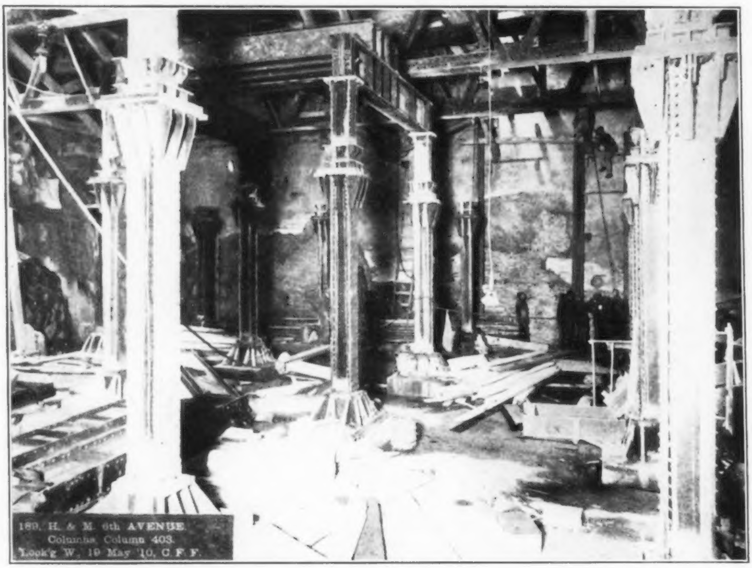


FIG. 2.—COLUMNS, WITH GRILLAGES, FOR SUPPORTING COLUMNS OF ELEVATED RAILROAD.

support. The three columns shored up were of a steel box, riveted type, 16 by 14 in. Rivets were removed from each column, and four steel brackets, aggregating 106 $\frac{3}{4}$-in. rivets per column, were riveted to them. Seven 36-in. girders, 39 ft. long, similar to those used in underpinning the elevated railway between 12th and 31st Streets, were placed on each side of each column. These girders, fourteen under each column, were supported at the outer ends by a pair of heavy 12 by 12-in. timber bents, parallel to the building front, in the excavation already made and 20 ft. from the building columns. At the inner ends they were supported by a reinforced beam or girder, built on the rock, as close as possible to the second line of building columns and parallel to the building front. These 39-ft. girders had a span of 34 ft. between the center of the bearings on the concrete girder and the center of the timber bent. Across each of the fourteen girder clusters, there was laid four 24-in., 100-lb. I-beams, two on each side of the column, on which rested the lower two brackets fastened to the building column. Above these I-beams, and parallel to the girders, there were twelve 24-in., 100-lb. I-beams, six on each side of each column, which were supported by the girders through timber blocking. These beams also supported the front wall of the basement. Across these I-beams there were laid four 24-in., 100-lb. I-beams, on which rested the upper two brackets of the building column. An additional support of the columns was afforded by using twelve suspender bolts (four 2-in. round and eight 1½-in. round), which held up the five 15-in. I-beams of the original grillage under the base casting of each building column. The top nuts of the bolts took bearing on the I-beams of the steel above. The proper distribution of the load over the parts of the steel underpinning was secured with wedges and the suspender bolts supporting the bases of the columns. The building columns were kept at their proper elevations with jacks and blocking under the ends of the girders.

After the building columns were picked up, the old foundations and the rock were removed, and, in this building, the columns were extended down to sub-grade with steel columns made up of three 24-in., 100-lb. I-beams, riveted together in the form of a T, and latticed and braced, as shown on Plate LXV.

After the excavation was completed, new grillages, made up of five 20-in., 80-lb. I-beams, 10 ft. long, were built under each column

at sub-grade and embedded in concrete; the extension columns were set in place, anchored to the new grillages, and bolted to the grillages of the old foundations. Then a concrete wall was laid with its outside face at the building line, filling the entire space of the excavation over the slope of rock. This wall extended up to the old foundations, which were then grouted on it, and, after the concrete had set, the shoring was removed and the subway station work at this point was prosecuted to completion.

The load on each of the columns of this building was calculated to be 400 tons. Of course, the loading on the wall columns was partly taken up by the brickwork of the north wall of the building, part of which was supported on the steel underpinning and part remained on the original foundation resting on rock.

The Browning-King Building.—The entire front of this building was supported by four cast-iron columns, 11 in. in diameter and 2 in. thick, two in the center and two embedded in the side-walls. The method of shoring this building was similar to that used in the Martin Building (concrete girder and timber bents), except that the details of the steel were different and the columns were not extended to sub-grade.

A similar concrete wall, filling the entire space of the excavation back of the building line, was built, the columns were grouted on top of this wall, and then the steel underpinning was removed.

The method of picking up the columns was as follows: As shown on Plate LXV, six holes were drilled completely through each cast-iron column, and then enlarged so as to pass 6 by 1¼-in. steel plugs. These plugs were 24 in. long, and were placed in the holes on edge, 18 in. apart, vertically.

At each end of each steel plug, two 6 by 4 by ¾-in. angles were bolted to give greater bearing on the I-beams supporting them. There was one 15-in., 60-lb. I-beam on each side of each column, supporting the steel plugs at each end, arranged in the form of towers. An equal bearing was maintained on these beams with blocking and steel wedges, so that the steel towers might be considered as one mass fastened to each column. The load was transferred through the 15-in. I-beams and blocking down to 36-in. girders similar to those used in the Martin Building. Ten girders, five on each side of the column, supported each of the two center columns; five girders and seven

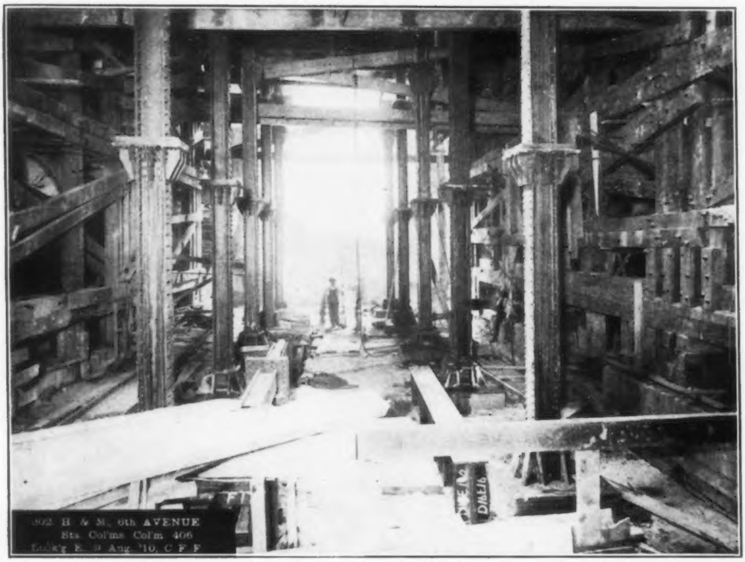


FIG. 1.—CLOSURE SECTION OF THIRTY-THIRD STREET STATION. STEEL COLUMNS SET UP.

FIG. 2.—CONCRETE FLOOR AND ROCK CORE BETWEEN SECTIONS OF THIRTY-THIRD STREET STATION.

24-in., 100-lb. I-beams supported the south wall and columns of this building and the north wall of the four-story building; and four girders and eight 24-in., 100-lb. I-beams supported the north wall and columns of this building and the south wall of the Bank Building. These girders had a span of 34 ft. between their bearings on the concrete girder and the timber bents. Additional support was obtained with four 2-in. round suspender bolts supporting six 9-in., 30-lb. I-beams placed under the base castings of the building columns, taking bearing at the bolt heads on the lower set of I-beams of the steel tower. These 9-in. grillages under the column bases were placed after the columns had been picked up on the steel towers, and were simply an additional precaution. After the removal of the shoring, the steel plugs were sawed off at the faces of the columns. The building columns were kept at proper elevation with jacks placed under the ends of the girders, as in the Martin Building.

The load on each of the columns of this building was calculated as between 240 and 260 tons.

The Bank Building.—As this building had no steel framing, a different method of support was necessary. The underpinning cared for the entire front of the building and a short length east of the corner. The foundation was composed of four piers or columns of granite-faced brick masonry. The front was carefully tied together with wire rope fitted with turnbuckles.

Owing to the condition of the excavation and the order of building the different pieces of station, it was found that the excavation in front of the bank could not be completed soon enough to allow sufficient time for the underpinning of the building and the subsequent construction of the piece of station opposite it. If the underpinning had been left until the excavation had been made in front of the building, the completion of the station and the opening of the tunnel for traffic would have been delayed. Owing to these conditions, two reinforced concrete girders were built on top of the rock, and the steel girders used in underpinning the building were supported on them. One of the girders was built inside the building and the other outside, and both were parallel to the building front. A concrete wall was then built in this trench up to within 2 ft. of the bottom of the steel girders supporting the building, and the foundations were carried

down with brick masonry to the top of the wall with the gradual removal of the girders.

At the outside edges of the piers of the foundation, recesses or steps were cut deep enough to allow the placing of one girder on each side of each pier. The remainder of each pier was then gradually removed, substituting the girders for the brickwork, until the piers were supported entirely on the girders. Eighteen steel girders were used in supporting this building, with the aid of the eight 24-in. I-beams and four girders used for the north wall of the Browning-King Building and the south wall of the Bank Building. The rock trench was line-holed along the inside face of the building and the face of the outside concrete girder, in order to provide an even break near the concrete girders, and the rock faces of the trench were braced with timber to prevent the rock from slipping.

Owing to the removal of the steam-heating plant of this building, which was originally under the sidewalk, it was necessary to erect a 50-h.p. boiler on the street to supply heat during the winter and until a new heating plant had been installed in the basement.

The four-story brick building between the Browning-King and Martin Buildings was supported on the underpinning used for the walls of the adjoining buildings.

A factor of safety of four was used in calculating the strength of the steel underpinning systems. The buildings were maintained to within $\frac{1}{4}$ in. of their original elevations by the jacks. The girders and I-beams were used several times, as the buildings were not all underpinned at the same time. There were no cracks or other damages due to settlement, and the interiors of the buildings were restored to the satisfaction of the owners.

Excavation.

The excavation was started in a series of pits to contain the elevated railway supports. These were usually 8 ft. wide, extended across the street the full width of the work, and were carried down to sub-grade in cases of dry soil and to within from 8 to 12 ft. of sub-grade in cases of wet soil, where piles were required to support the elevated railway.

The excavation of the cores of earth or rock under each pair of bents of the elevated railway were generally from 100 to 110 ft. long,

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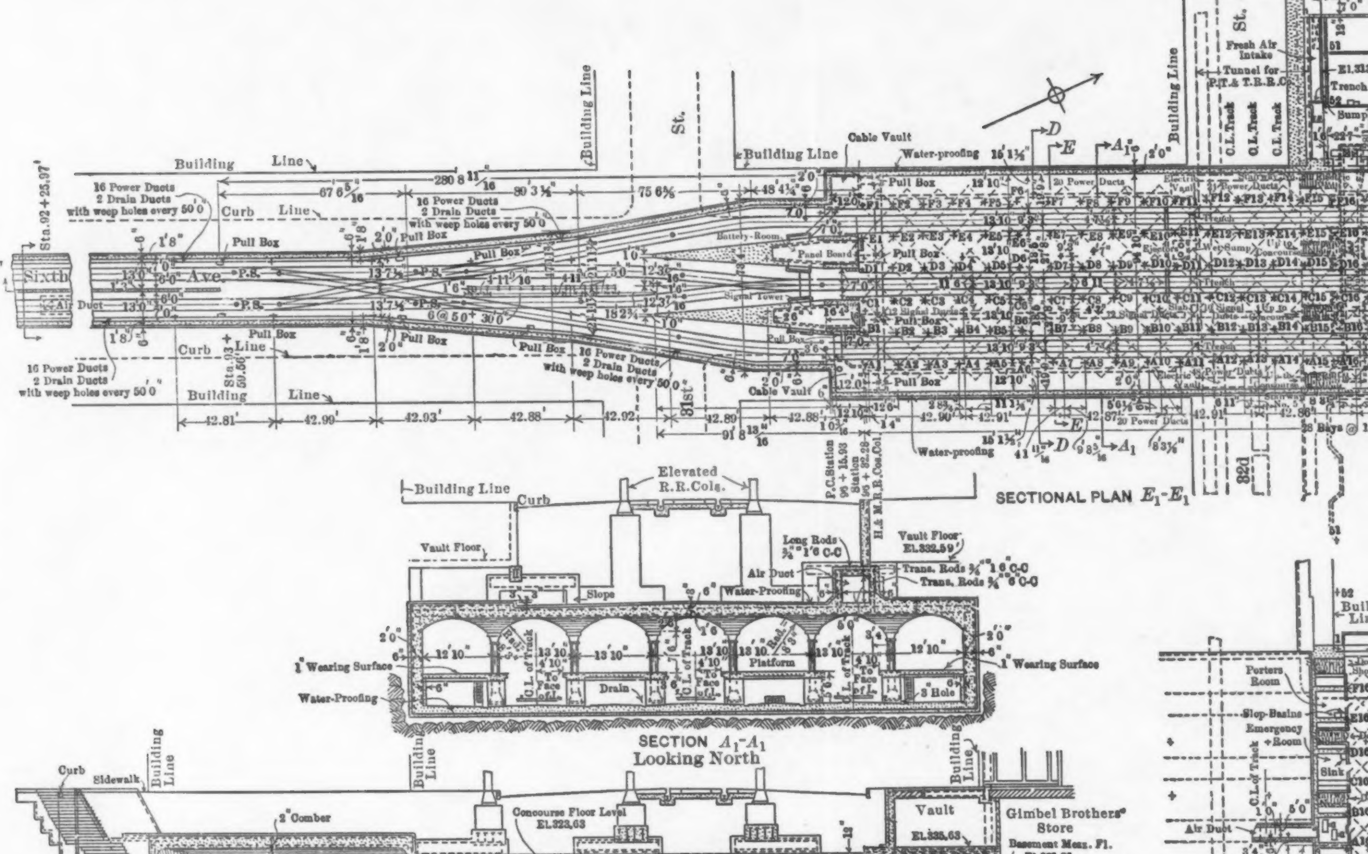
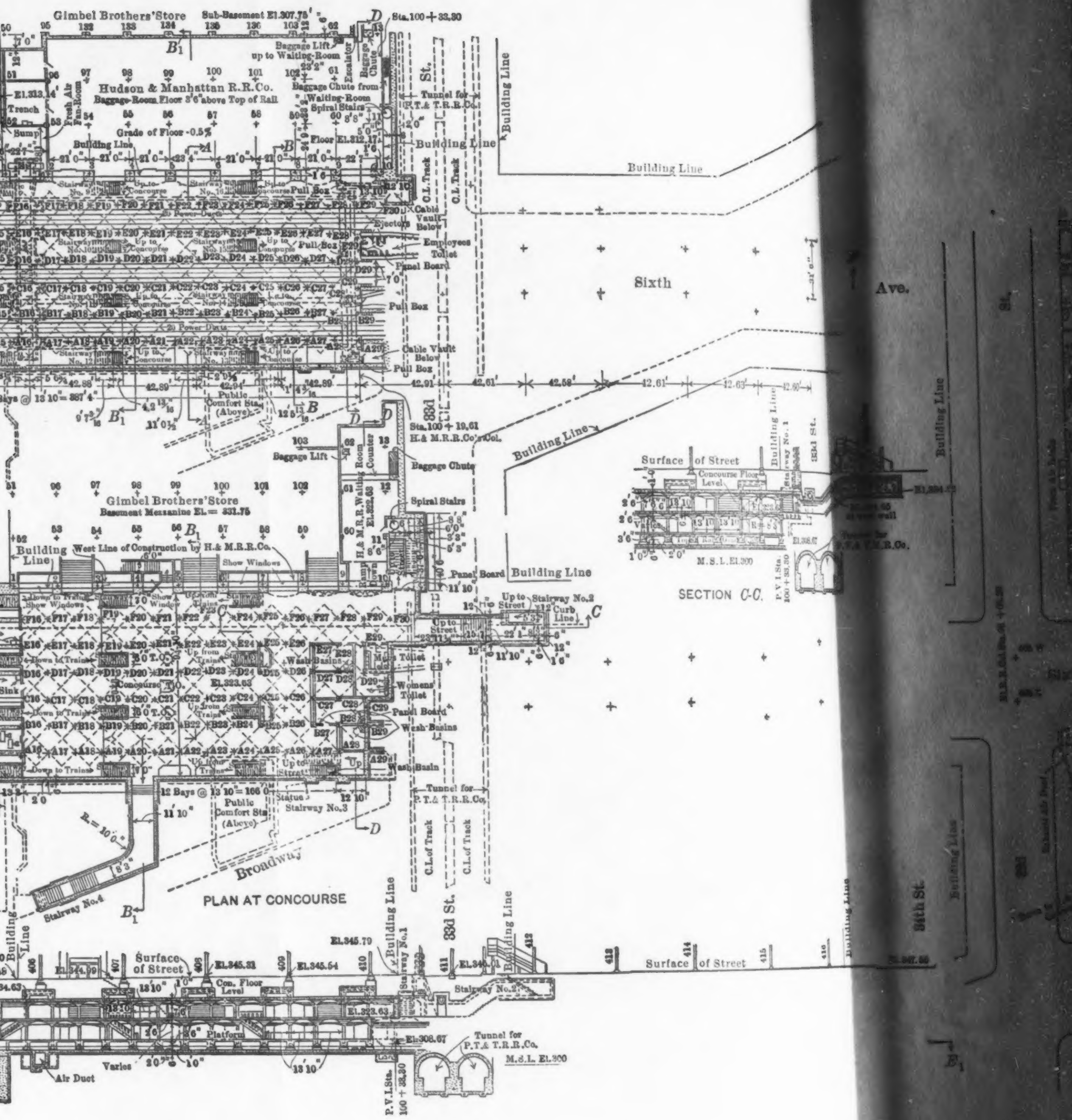


SECTIONAL PLAN E_1-E_1

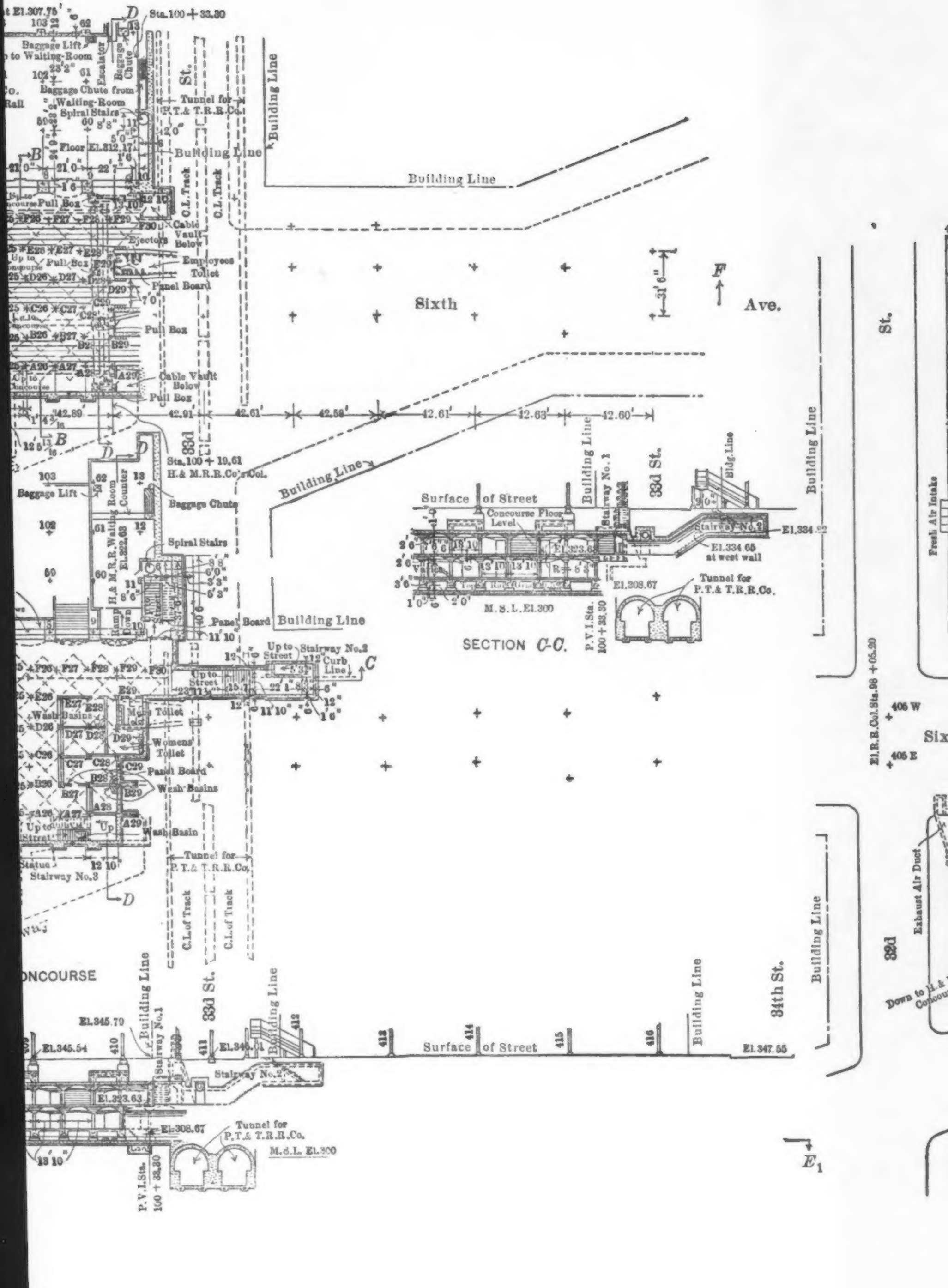
SECTION A_1-A_1
Looking North

SECTION B_1-B_1
Looking South

SECTION F-F

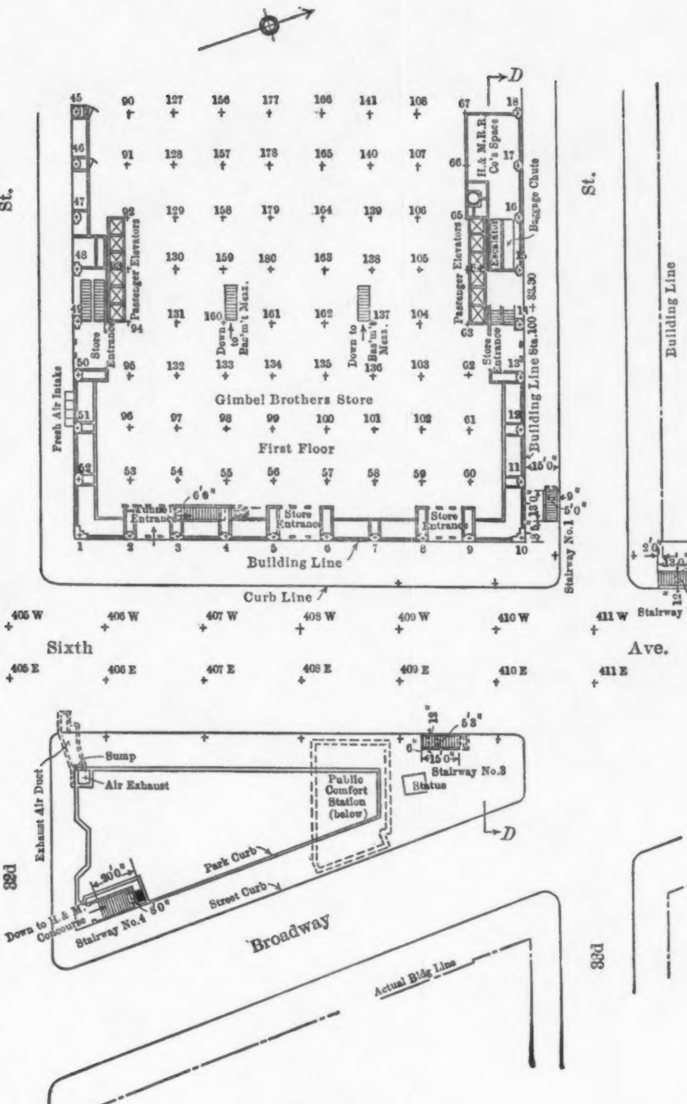


Gimbel Brothers' Store
Hudson & Manhattan R.R.Co.
Sixth
Ave.
SECTION C-C.
Gimbel Brothers' Store
PLAN AT CONCOURSE
Broadway
33d St.
34th St.
Surface of Street
Building Line

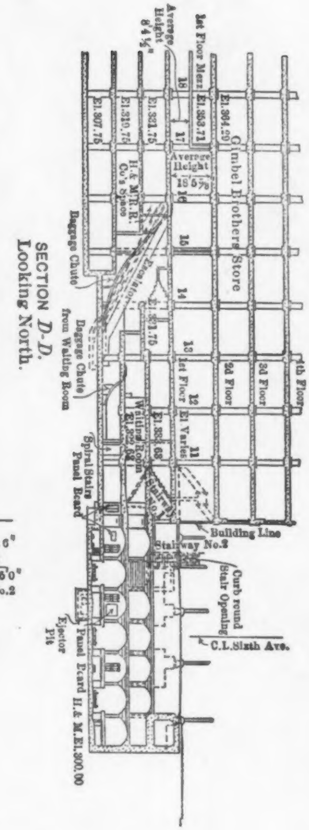


Sta.100 + 33.30
Baggage Lift
Escalator
Baggage Chute
Baggage Chute from Waiting-Room
Spiral Stairs
Floor El.312.17
Tunnel for P.T.& T.R.R.Co.
Building Line
C.L.Track
Cable Vault Below
Ejectors
Employees Toilet
Panel Board
Pull Box
Cable Vault Below
Sixth
Ave.
33d
St.
Sta.100 + 19.61
H.& M.R.R.Co's Col.
Baggage Chute
Spiral Stairs
H.& M.R.R.Waiting Room
El.322.63
Counter
Ramp
Panel Board
Up to Street
Stairway No.2
Curb Line
Mens Toilet
Womens Toilet
Wash Basins
Wash Basin
Statue
Stairway No.3
CONCOURSE
Surface of Street
Concourse Floor Level
Stairway No.1
Stairway No.2
Bldg. Line
El.334.65 at west wall
El.308.67
Tunnel for P.T.& T.R.R.Co.
M.S.L.El.300
P.V.I.Sta. 100 + 33.30
SECTION C-C.
33d St.
34th St.
El.345.79
El.345.54
El.323.63
El.308.67
El.347.55
M.S.L. El.300
Fresh Air Intake
Exhaust Air Duct
Down to H.& M. Concourse
El.R.R.Col.Sta.98 + 05.20
405 W
405 E
Six

PLATE LIX.
PAPERS, AM. SOC. C. E.
AUGUST, 1912.
BURROWES ON
THE SIXTH AVENUE SUBWAY,
H. & M. R. R.



SURFACE PLAN



HUDSON & MANHATTAN R.R.CO.
33D STREET STATION
GENERAL PLAN & PROFILE

from north to south, depending on the depth, and were of the full width required for building the subway and including the sewers. When these large pits were finished to sub-grade, the lengths of subway were built in them, and the elevated railway columns were set in their final position on the roof. The intermediate cores of excavation between these large pits were then removed to sub-grade, thus completing the excavation.

The timbering is shown on Plates LXII and LXV. There were two or three lifts in the sheeting of the cuts, depending on the depth of the excavation and the elevation of the sewers alongside the subways. Where the soil was wet the lower lift was made up of 3-in. tongued and grooved, yellow pine sheeting. The sheeting for dry soil was of 2-in. yellow pine.

The bracing of the cut, as shown on Plate LXII, was designed so as to interfere as little as possible with the laying of the subway concrete, but the braces were left in place and allowed to pass through the concrete roof where there was any advantage in the saving of time or cost. The bents or timber sections were made up of 10 by 10-in. rangers and 12 by 12-in. braces and posts, and were generally placed 10 ft. apart. The details of the timbering are shown on Plate LXII, and need no special description.

The temporary planking of the street surface was laid on 10 by 10-in. and 12 by 12-in. stringers or deck pieces running longitudinally with the work, and on them was placed 4- and 5-in. planking laid transversely across the stringers. This deck or planking was supported by the posts of the cut bracing. As this planking was necessarily in service a long time, and subjected to the wear of the street traffic, it was necessary to watch it constantly and renew it in order to avoid accident.

When the 39-ft. girders were used in underpinning the elevated railway columns, as shown on Plate LXII, the timbering of the cuts was greatly simplified during the rock excavation between stations by using six 24-in., 80-lb. I-beams, 35 ft. long, in three pairs, each pair supported by one 12 by 12-in. post under the ends, resting on the rock. These I-beams supported the street surface and permitted the elimination of the posts necessary to support it. The bracing of the sheeting was carried across the cuts as usual, but required very few posts.

The materials through which the excavation was made were generally dry sand and loam, but between 12th and 13th Streets a fine sand with water was encountered at sub-grade, below tide level, and it was necessary to lay a 10-in. timber floor in the bottom of the cut on which was laid a 3-in. planking to furnish a proper foundation on which to build the tunnel. It was also necessary to provide temporary drains to carry off the water while laying the concrete in this part. Between 15th and 18th Streets a wet sand was encountered at sub-grade, but in this case it was simply necessary to put in a floor of 3-in. planking on which to lay the concrete floor. A small outcrop of rock was removed at sub-grade at 14th Street, but the rock excavation really commenced at 20th Street and continued north throughout the work to 33d Street. The rock varied in depth from 6 ft. above sub-grade at 20th Street to 35 ft. above sub-grade at 33d Street, and was very uneven, at times running just below sub-grade, and requiring a deeper excavation at these points.

Rock Excavation.—The rock formation was the usual mica schist, the general slope of the strata being about N. 30° W.; the inclination was between 15° and 20° east of vertical.

The bench rock was drilled with "slugger" drills and 2-in. drills, and the boulders and large pieces with "Little Wonder" drills. It was blasted and broken into small pieces suitable for dumping into the trucks. The drill holes in the bench rock were generally 14 ft. deep. Where the temporary foundations of the elevated railway columns were on the rock, the excavation between these columns was carried as close to them as safety allowed, and lines of holes were drilled and blasted transversely across the cuts so as to make an even break and give as much space as possible in the excavated pits for building the pieces of tunnel or station. The rock excavation was carried toward the east on account of the slope of the strata.

From 31st to 33d Streets, and the 33d Street Station.—In this section the methods of excavation were similar to those described for the work between 12th and 31st Streets, excepting that, on account of the larger pieces or lengths of station required to be built in the first operation, as described in "Underpinning the Elevated Railway," the method of underpinning the elevated railway made the pits of shorter length from north to south.

FIG. 1.—WALL AND ROOF FORMS, THIRTY-THIRD STREET STATION.

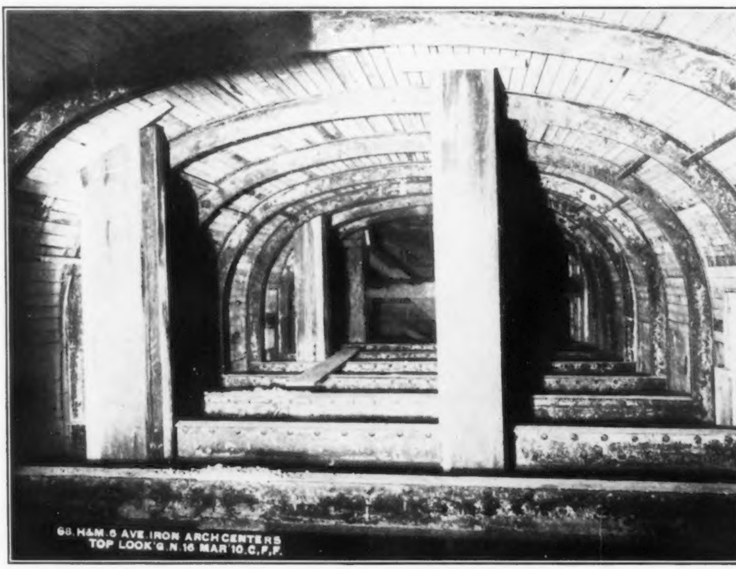


FIG. 2.—STEEL RIBS AND LAGGING OF FORMS IN STANDARD SUBWAY SECTION.

As shown on Plate LXII, the timbering and support of the street between 31st and 33d Streets differed from that of the work from 12th to 31st Streets. Light steel lattice trusses, 8 ft. deep, were erected under the street planking and the surface car tracks, pipes, electric properties, and the street surface planking were all supported on these trusses. This eliminated all the obstructions caused by the usual method of bracing the cuts with timber, and furnished a clear space in which to build the subway or pieces of station under the columns. As shown on Plate LXV, each lattice truss was made up of a bottom chord, two 8 by 8 by $\frac{11}{16}$-in. angles and one 7 by $\frac{9}{16}$-in. web-plate between the $\frac{9}{16}$-in. gusset-plates connecting the diagonals to the chord. The top chord consisted of two 8 by 8 by $\frac{1}{2}$-in. angles and one 16 by $\frac{9}{16}$-in. web-plate. The bracing or diagonals were made up of two angles, 6 by 4-in. and 4 by 3-in., from $\frac{3}{8}$ to $\frac{3}{4}$ in. thick. These trusses were made up in small pieces in order to facilitate their erection under the street planking. When the lengths of subway or sections of station had been built under these trusses, and the elevated railway columns had been set on their final foundations, the posts for the street pipes, street surface, etc., were carried down to the roof of the completed work. The trusses were then removed. After the columns, resting on temporary concrete foundations, were again underpinned, the foundations and cores of rock under them were removed.

Construction.

Concrete.—The concrete used on the work was mixed on the street surface in Ransome, $\frac{3}{4}$-yd. mixers. The proportions of concrete for foundations, walls, and roofs was 1 to 6, varying from 1 cement, 2 sand, and 4 broken stone to 1 cement, $2\frac{1}{2}$ sand, and $3\frac{1}{2}$ broken stone, according to the voids in the sand and stone. The concrete used for track floors on bed-rock, and for other floors or places where there was little strain or load, was composed of 1 cement, 3 sand, and 5 stone.

The stone was $\frac{3}{4}$ and 1-in. and run-of-crusher Hudson River limestone. It was found that the $\frac{3}{4}$-in. stone first used contained such a large percentage of screenings and dust that it was advisable to get 1-in. stone, ranging from $1\frac{1}{4}$ to $\frac{1}{2}$ in., with a small percentage of screenings and a very small percentage of dust. This stone was adopted, for the concrete work throughout, as being better for a

1 to 6 mixture, under the conditions of the work. The stone and sand were delivered alongside the dock at the yard, 42d Street and East River, and stored there. The stone used in the street restoration, being required in small quantities, was delivered directly on the work.

The sand was obtained mostly from Hempstead Harbor, Long Island; it was of a good coarse quality and contained from 2 to 3% of loam. The maximum quantity of loam allowed was 2%, and, during the first two years of the work, it was very hard to obtain sand of this quality, but there was no difficulty during the last three years.

"Giant" cement was used throughout the work. It was tested at the company's laboratory in Jersey City, and delivered at the yard in trucks.

The cement was tested according to the Am. Soc. C. E. standard. It was uniform in quality, and there was no difficulty in obtaining a reliable set or proper age.

The water for the concrete was obtained from the city mains through metered connections at convenient points along the work. The mixers were run with compressed air obtained from the company's power plant at the 42d Street yard. The mixing gangs consisted of from 14 to 18 men, as follows:

One mechanic running the mixer,
One skilled laborer dumping the cement and handling the water,
From 6 to 8 men loading wheel-barrows, and feeding sand and stone to the mixer,
From 6 to 8 laborers delivering concrete from the mixer at the point of laying.

The sand and stone were measured and delivered to the mixer in wheel-barrows. The concrete was carried from the mixers to the points of laying in "Ransome" concrete carts, and dumped into chutes at the street surface.

The sizes of the gangs placing the concrete varied considerably with the number of mixers supplying concrete and with the size of the structure, the difficulties in placing, and the length of haul between the mixer and point of laying. All the concrete was mixed "wet" and of such consistency that it would flow readily through the chutes and be handled easily. The length of haul of the concrete

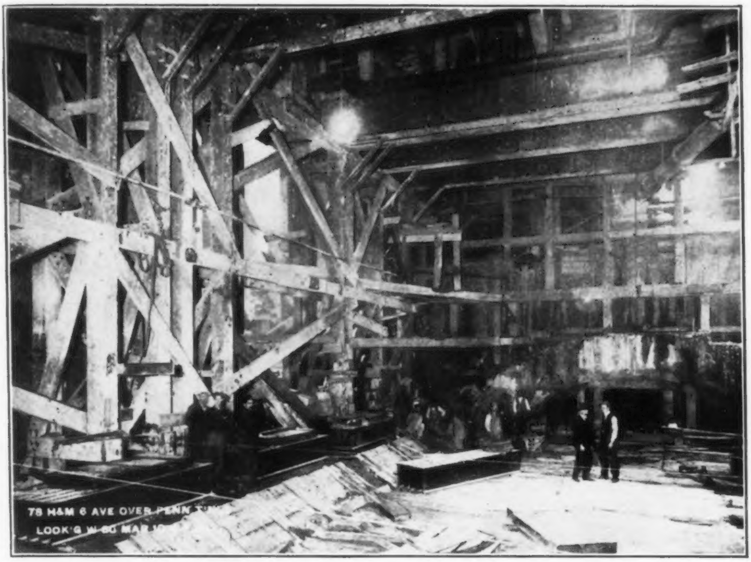


FIG. 1.—PLACING STEEL GRILLAGES FOR SUPPORT OF GIRDERS OVER ARCH OF PENNSYLVANIA RAILROAD TUNNEL.

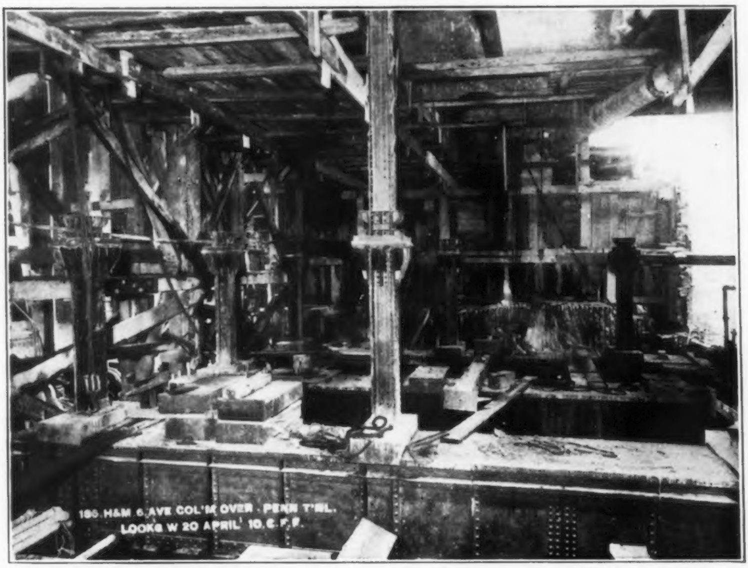


FIG. 2.—STEEL CROSSING OF PENNSYLVANIA RAILROAD TUNNEL.

was from one to two blocks, or from 260 to 520 ft. There was little delay in delivering the concrete from the mixer to the points of laying, because the men could wheel the carts over almost any portion of the street surface.

Owing to the great number of rods in the reinforcement, it was difficult to handle the concrete in the forms for the roofs and walls, and great care had to be exercised not to displace the rods or crowd them against the forms.

All the concrete of the subway was laid under the street planking, and great care was necessary to prevent dirt or chips from being dropped into the forms. No face mixture was used; the concrete was simply rammed or "quaked" by the use of long 2 by 4-in. timbers in the wall work and rammers in the floor and roof work. The faces of the walls and roofs showed very few voids or dry spots after the removal of the forms. The form centers in the standard tunnel section were composed of steel ribs made up of two 5 by 3-in. steel angles. These ribs extended from the toes of the center- and side-walls up to and including the arches. The inside face planking of the walls and the arch lagging were fastened to them with wire staples. The inside wall face forms were of 2 by 10-in., tongued and grooved spruce, and the arch lagging of beveled spruce 3 in. thick.

In building the groined arch forms of the station roofs, the ribs were placed under the groin lines of the arches and were of 2-in. spruce supported by 4 by 4-in. spruce posts. The lagging was beveled, planed, yellow pine, with butt joints, and was nailed to the spruce ribs. The face planking and lagging were used over again as often as their condition allowed, but no attempt was made to redress the forms, as there was little economy in it, there being plenty of places where the rough forms were good enough in the exposed faces of the work.

It is a feature of this concrete work that the forms failed or slipped only four times during the entire work.

The concrete was under the most favorable conditions for setting, as it was under the street planking, thus avoiding sun drying and quick setting and being aided by the drafts of cool moist air circulating through the cuts and finished pieces of tunnel.

The concrete of the subway and stations was laid up in three lifts: first, the floors and foundations; second, the walls from the founda-

tions to 1 ft. below the springing lines of the arches; and third, the roofs.

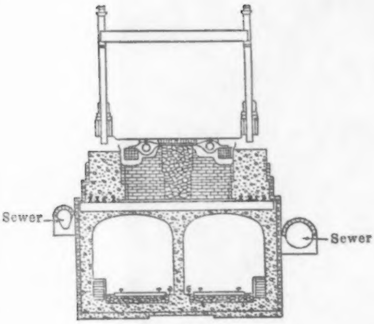
Concrete was not laid when the temperature was lower than 25° Fahr. and in all cold weather the concrete was covered with salt hay to prevent freezing. Salt was not used in mixing the concrete. The temperature in the pits was very seldom lower than 30° Fahr.

The forms were dressed with soap or paraffin oil in all the work, excepting the arches of the 28th and 33d Street Stations, where the dressing with oil or soap was abandoned in order to obtain a better bond between the white cement finish and the concrete in whitewashing the interiors of the stations.

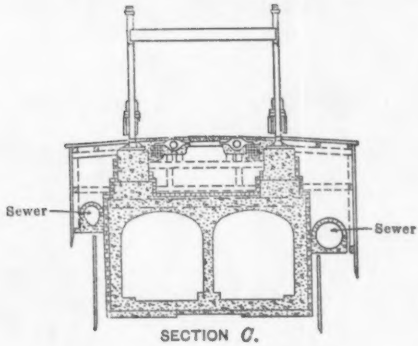
The steel and cast-iron columns of the stations were filled with 1:3 mortar grout poured in from the tops; this was done in two operations in order to permit the first batch to settle.

The spaces between the girders spanning the Pennsylvania Tunnel at 32d Street were grouted with neat cement. Mortar could not be used in this grouting because the spaces between the angle stiffeners were so small that mortar was too thick to form a certain enough grout. In this work the cement grout was poured into a 2-in. pipe at the street surface, which gave a head of 35 ft. for the pressure required to fill completely the spaces between the girders.

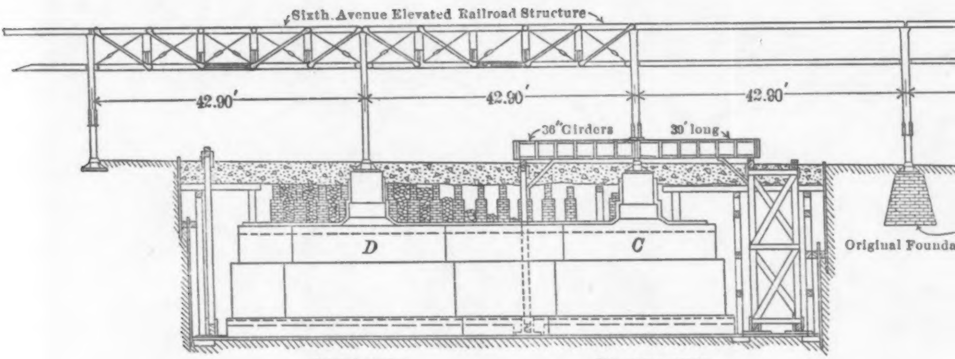
Reinforcing Rods and Expanded Metal.—The reinforcing rods were 1-in. and $\frac{3}{4}$-in. square Ransome twisted steel for the walls, roof, and foundations, and $\frac{3}{8}$ and $\frac{1}{2}$ in. for the platforms and thin walls. All the rods were delivered alongside the company's dock in lengths ranging from 10 to 30 ft. Rods were bent to special shapes in the yard, by using a bender made on the work and having a hydraulic jack. The rods requiring a bend up to about 45° from the straight were bent by hand. All the lengths and shapes represented certain positions in the walls, roofs, etc. The plans showed continuous rods around the shell of the subways and stations, and longitudinally, so that it was necessary to break the rods at points where it would do the least harm, and bond the breaks by an overlap of 40 diameters of the rods. The rods of the walls, roof, etc., were held 2 in. from the forms by using small mortar blocks laid between them and the forms. The rods in the roof were supported by the forms and by wire hangers fastened to timbers above. The weight of rods per cubic yard of concrete in the station roofs was very high, and the weight of the



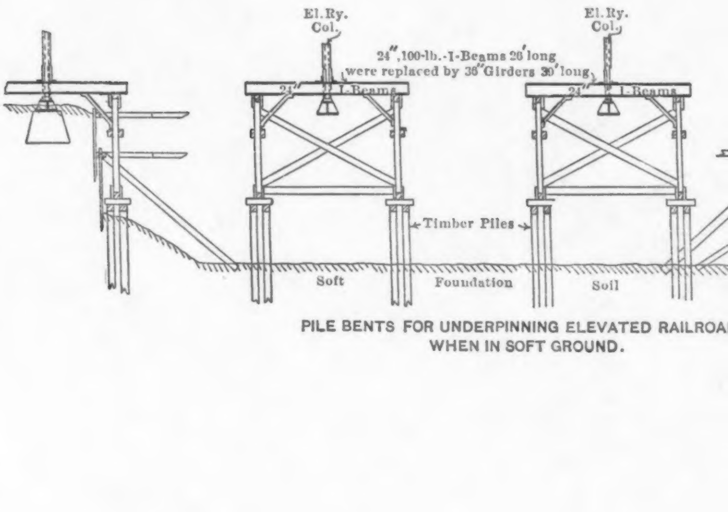
SECTION *D.*



SECTION *C.*

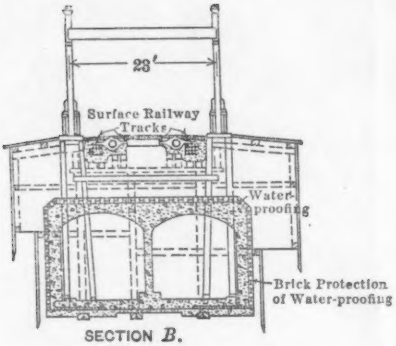


FIFTH STEP. FOURTH STEP.

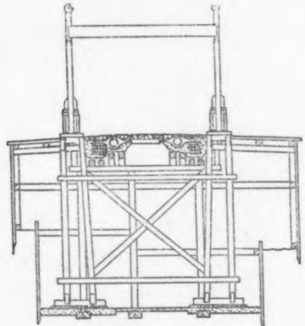


PILE BENTS FOR UNDERPINNING ELEVATED RAILROA
WHEN IN SOFT GROUND.

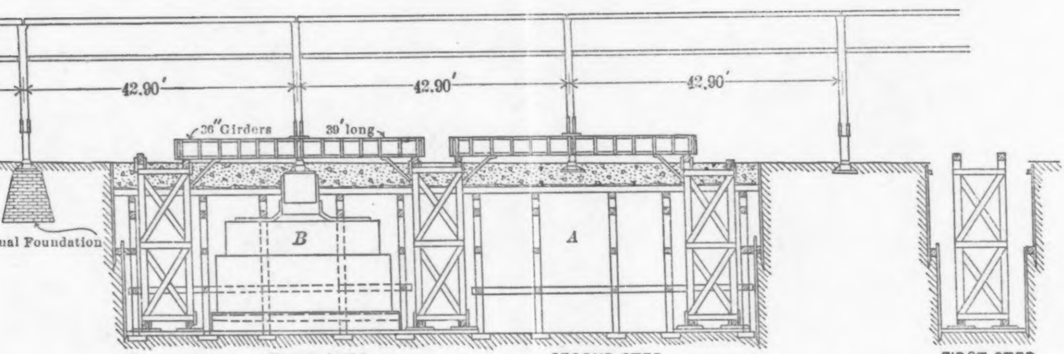
PLATE LXII.
PAPERS, AM. SOC. C. E.
AUGUST, 1912.
BURROWES ON
SIXTH AVENUE SUBWAY,
H. & M. R. R.



SECTION *B*.



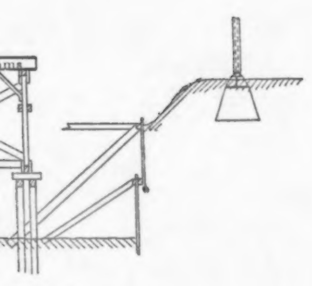
SECTION *A*.



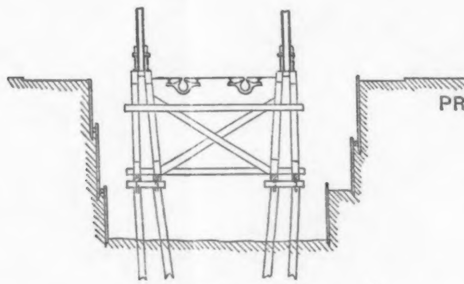
THIRD STEP.

SECOND STEP.

FIRST STEP.
Timber Bents for Underpinning
Elevated Railroad.



RAILROAD



SECTION OF PILE BENTS.

PROGRESSIVE STEPS & METHODS
12TH TO 31ST STREETS.

concrete and rods combined required that the forms of the roofs be well supported by the 4-in. posts. The average spaces between the rods through which the concrete was poured was 12 by 12 in., and particular attention was given to spacing the rods properly so that the concrete could be placed in the most economical way. The clusters of rods (in the form of baskets) over the station columns were especially difficult to place and keep in position, because the concrete dumped into the haunches of the arches tended to shift the entire cluster so that the rods were displaced or forced against the forms.

Expanded metal or wire cloth was laid under the roof beams of all the flat arches to bond the concrete cover of the beams to the concrete between them.

Brickwork.—The bricks used in underpinning buildings, in the foundations of the surface railway, and in the stations, were the usual standard house brick, and were laid in 1:3 mortar. The partition walls of the stations, booths, ticket offices, etc., were built of 3-in. and 4-in. hollow tile.

Water-Proofing.—The entire work between 12th and 27th Streets was water-proofed with "Siastex," a burlap and asphaltum process. A strong and durable burlap, of more practical value than the usual felt, was used because of the necessity of handling the reinforcement rods over it on the floors and walls. The normal thickness of the water-proofing was 2-ply with felt "plus ply." This was increased to 3- and 4-ply where water pressure and flow were met. A single ply of felt "plus ply" was first laid under all the water-proofing where the concrete surfaces were wet, and over this were laid the burlap and tar. On the roof of the 23d Street Station and under the floor of the enlargement at 12th Street, where there was a flow of water and very wet conditions, a brick and mastic was used. This consisted of a tarred felt "plus ply," on which was laid hot bricks which were grouted with pitch. In all the water-proofing in which the burlap was used, an armor course of 3-in. hollow tile or 4-in. brick was laid to protect it. In the floors, a 6-in. concrete floor was laid, on which the water-proofing was placed. Between 27th and 33d Streets, a new process of water-proofing was tried. The great cost of the membrane water-proofing called attention to the possibility of using a compound to be mixed with the concrete with a great saving in cost. Therefore a powder compound was mixed with the concrete in this section.

It was anticipated that the shrinkage of the concrete in drying would open up cracks which would require caulking, but the saving over the cost of the previous method was calculated to pay for this and still effect considerable economy.

Tunnel Ducts.—The ducts in the subway for the cables of the power and light services were laid on the toes of the side-walls and under the platforms of the stations. These ducts were laid after the completion of the shells of the subway and stations, and call for little description as no difficulties were met.

All the ducts were of vitrified clay, usually of 4-hole pieces, 30 in. long, with the addition of 2-hole pieces in certain places. The joints were wrapped with canvas, and two dowel pins were placed in each duct joint. The joints were made with 1:3 mortar. Bond straps of $\frac{1}{8}$ by $1\frac{1}{2}$-in. iron were laid between the ducts to bond the concrete faces to the ducts. Manholes were built in each subway tube, between stations, and at each end of the stations for pulling and splicing purposes. The duct banks were covered with a concrete jacket to protect them from injury, and the faces of this jacket were reinforced with expanded metal or wire cloth.

Sewers.—Previous to the commencement of the subway work, a main outlet sewer 5 500 ft. long was built in 18th Street from Sixth Avenue to the North River, commencing at Sixth Avenue as a brick structure 5 ft. 9 in. in diameter. At Eleventh Avenue it is a box-shaped section, of reinforced concrete, and is finished at the river end with two barrel sewers supported on the timbers of the steamship pier at West 18th Street.

The original sewers in Sixth Avenue were in the center of the street, and, as they were below the level of the roofs of the subway and stations, the entire system, from 12th to 33d Streets, had to be rebuilt. The old sewers were broken away as the work progressed, and the flow of sewage was maintained through temporary 12 to 20-in. spiral riveted pipes. In rebuilding, it was necessary to provide a sewer on each side of the subway, and to connect those in the side streets.

The new system in Sixth Avenue consists of a trunk sewer on the east side, flowing north from 14th Street to the sewer junction under the subway at 18th Street, 3 ft. 6 in. in diameter at 14th Street, and 4 ft. in diameter at the junction; and a trunk sewer from 31st

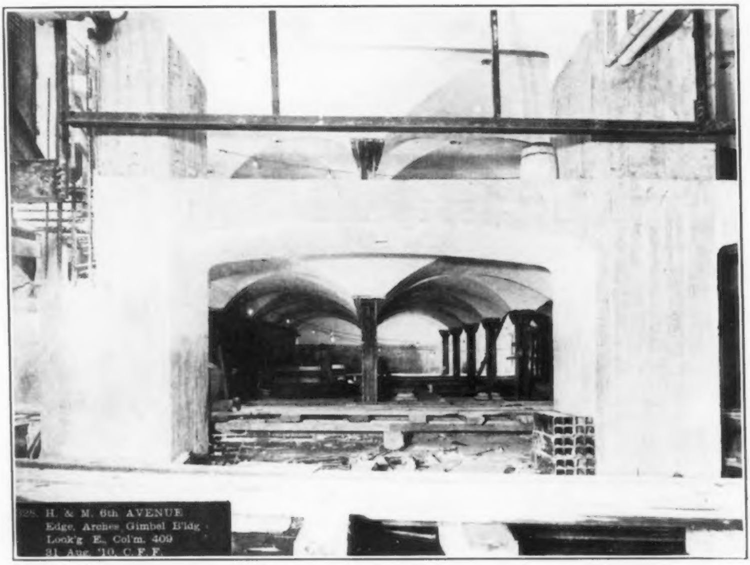


FIG. 1.—TRAIN FLOOR AND CONCOURSE OF THIRTY-THIRD STREET STATION.

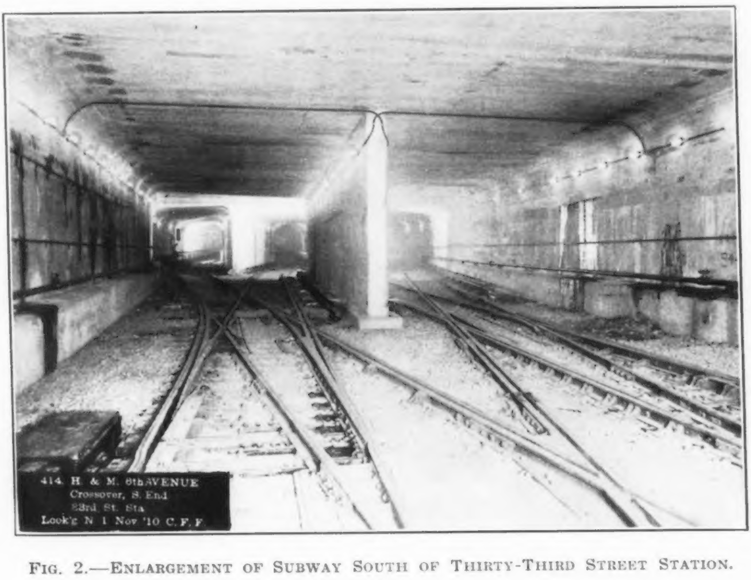


FIG. 2.—ENLARGEMENT OF SUBWAY SOUTH OF THIRTY-THIRD STREET STATION.

PLATE LXIV.
PAPERS, AM. SOC. C. E.
AUGUST, 1912.
BURROWES ON
THE SIXTH AVENUE SUBWAY,
H. & M. R. R.

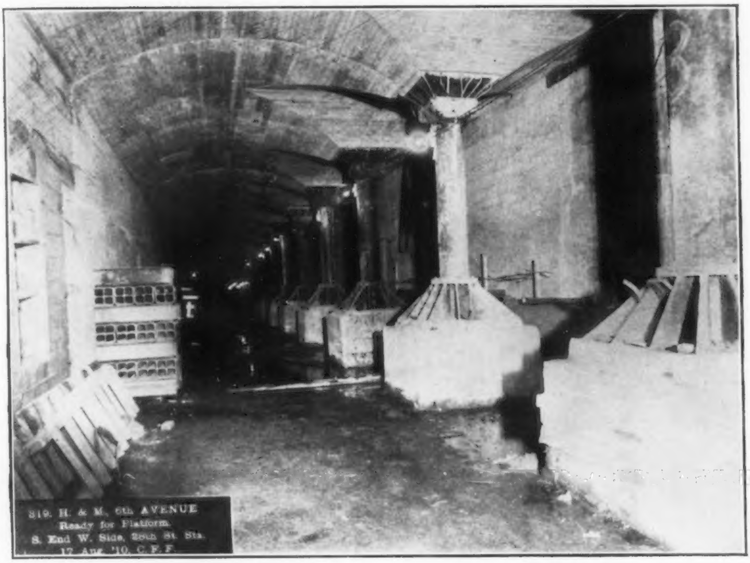


FIG. 1.—STANDARD STATION BEFORE PLATFORM WAS BUILT.

FIG. 2.—FORMS FOR GROINED ARCHES OF ROOF, THIRTY-THIRD STREET STATION.

Street south to the junction at 18th Street, 3 ft. 6 in. in diameter at 31st Street and enlarging to 4 ft. 9 in. at the junction, running down the east side from 31st Street to 21st Street, passing under the 28th and 23d Street Stations and crossing under the subway south of 21st Street to the west side where it turns south to the junction at 18th Street. Smaller laterals, ranging from 12-in. vitrified pipe to 3 ft. 6-in. by 2 ft. 4-in. egg-shaped sewer, were built on both sides of Sixth Avenue at the points not served by the trunk sewers.

The pipe sewers were laid on a concrete invert or foundation 8 in. thick; all the other sewers were built with 8 to 12-in. concrete inverts and 8-in. brick crowns. The forms for the concrete invert were of pine ribs with 1-in. lagging, or were collapsible steel centers. The house connections were of vitrified or cast-iron pipe, and each sewer connection was made with a cast-iron spur.

Surface Railway.—The surface railway track structure was supported on the bracing and posts of the excavation during construction, and brick piers for heights up to 7 ft., or reinforced concrete bents for greater heights, were built up from the subway and station roofs every 5 ft. under each yoke. The tracks were then jacked up to proper grade and the piers finished under the yokes.

Owing to the settlement of the track structure, a large part of the electric duct benches of the surface railway had to be cut away and rebuilt. There were no accidents during the temporary support of the surface tracks excepting a few "blow-outs" of the power cables which were injured because of their proximity to the work.

Street Restoration, Pipes, etc.—At the commencement of the work there were gas mains, water mains, and electric duct banks, including those for the street railway, and all these had to be relaid. There were also additions laid under agreements with the owners, for which the work was compensated.

All the water and gas mains were supported by 10 by 10-in. timber bents where the excavation was carried under them. These bents were erected on solid earth or rock, or on the roofs of the stations, and were built every 6 ft. on both sides of the subway. During the subway work these pipes, duct banks, and cables (where the ducts were stripped from them so that they could be raised outside the lines of the work) were supported by the bracing and posts of the excavation. Before the general excavation was started, 6, 8, and 16-in.

wrought-iron pipe by-passes were laid at the curb lines for the temporary supply of gas, and then the gas was removed from the mains. The empty mains were slung to the timbers and left until they were relaid in their new positions in the street, at which time the house connections were replaced on the mains and the temporary by-passes were removed. The traffic required that only one side of the street be opened at a time in any block.

The pipes were laid on the timber bents and the duct benches on the rammed fill, excepting those of the surface railway which were laid on the brick or concrete piers and the dry packing laid between the piers.

When the gas and water services were replaced in the mains, the street was back-filled and the temporary pavement laid and maintained until the fill had settled sufficiently for the laying of the final asphalt pavement.

This temporary pavement was of granite blocks which had been removed from the street when the work was commenced.

Storage Yards.

A yard and storehouses were provided at 17th Street and Tenth Avenue for the work between 12th and 27th Streets, and at 42d Street and East River for the work between 27th to 33d Streets. In these yards were the usual storehouses and shops for the storage of materials and plant. The storage of materials in the side streets was limited to 100 ft. on each side of Sixth Avenue. All the cast iron and most of the steel was stored in these yards, in addition to the rods, timber, concrete materials, etc. The two yards were similar, but that at 42d Street contained the power-house for the supply of compressed air.

The haul from the 17th Street yard was from ¾ to 1 mile, and that from the 42d Street yard was from 1½ to 2 miles. The latter yard had the advantage of a dock on the East River at which practically all the materials were delivered, consequently the cost was less than for those delivered at the other yard, which had no dock.

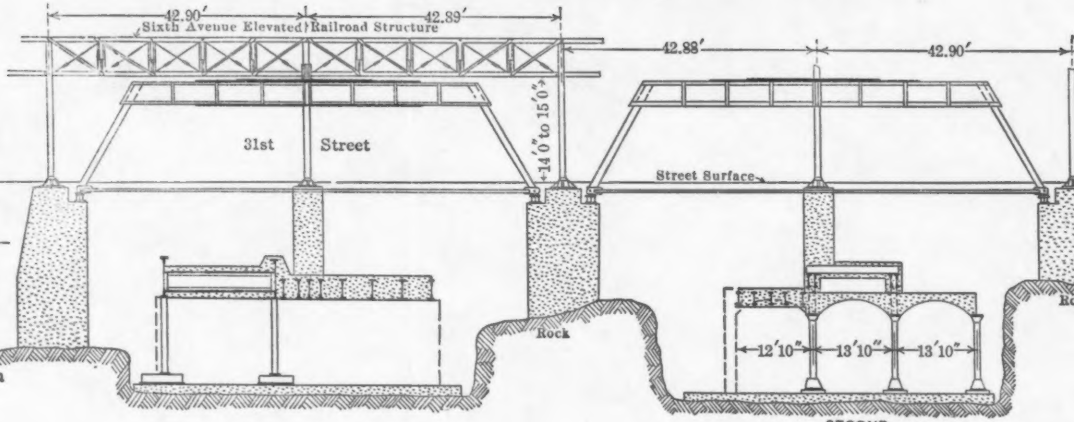
Some of the items which secured economy at the 42d Street yard were as follows:

The use of derricks for unloading material from scows;

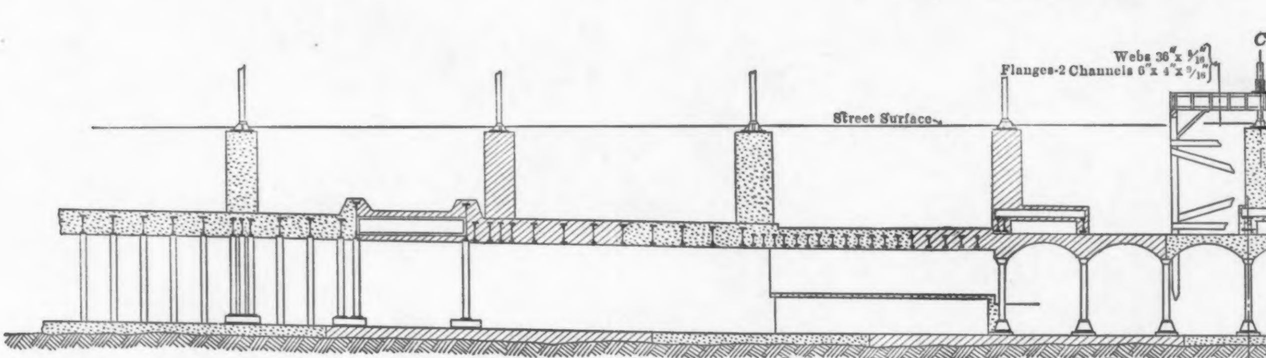
The rapid unloading of scows, thus saving demurrage;

FIRST STEP.

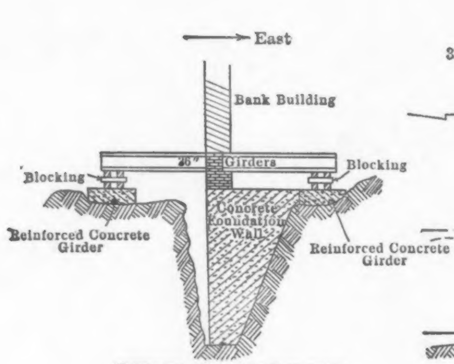
Carrying Elevated Railroad Column Foundation Down to Rock.



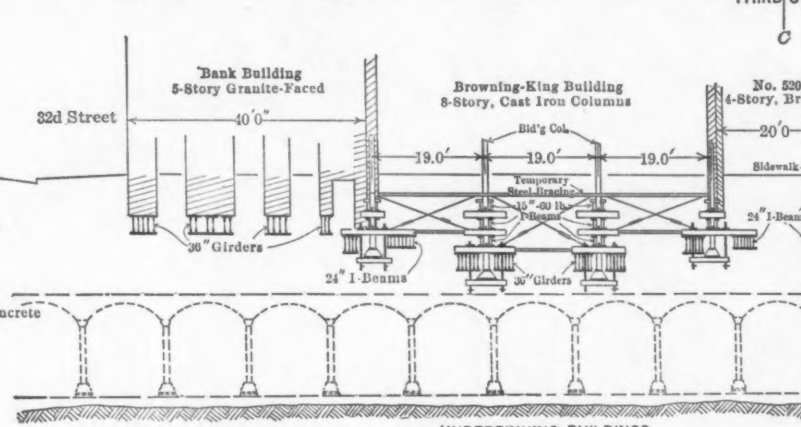
SECOND



THIRD S

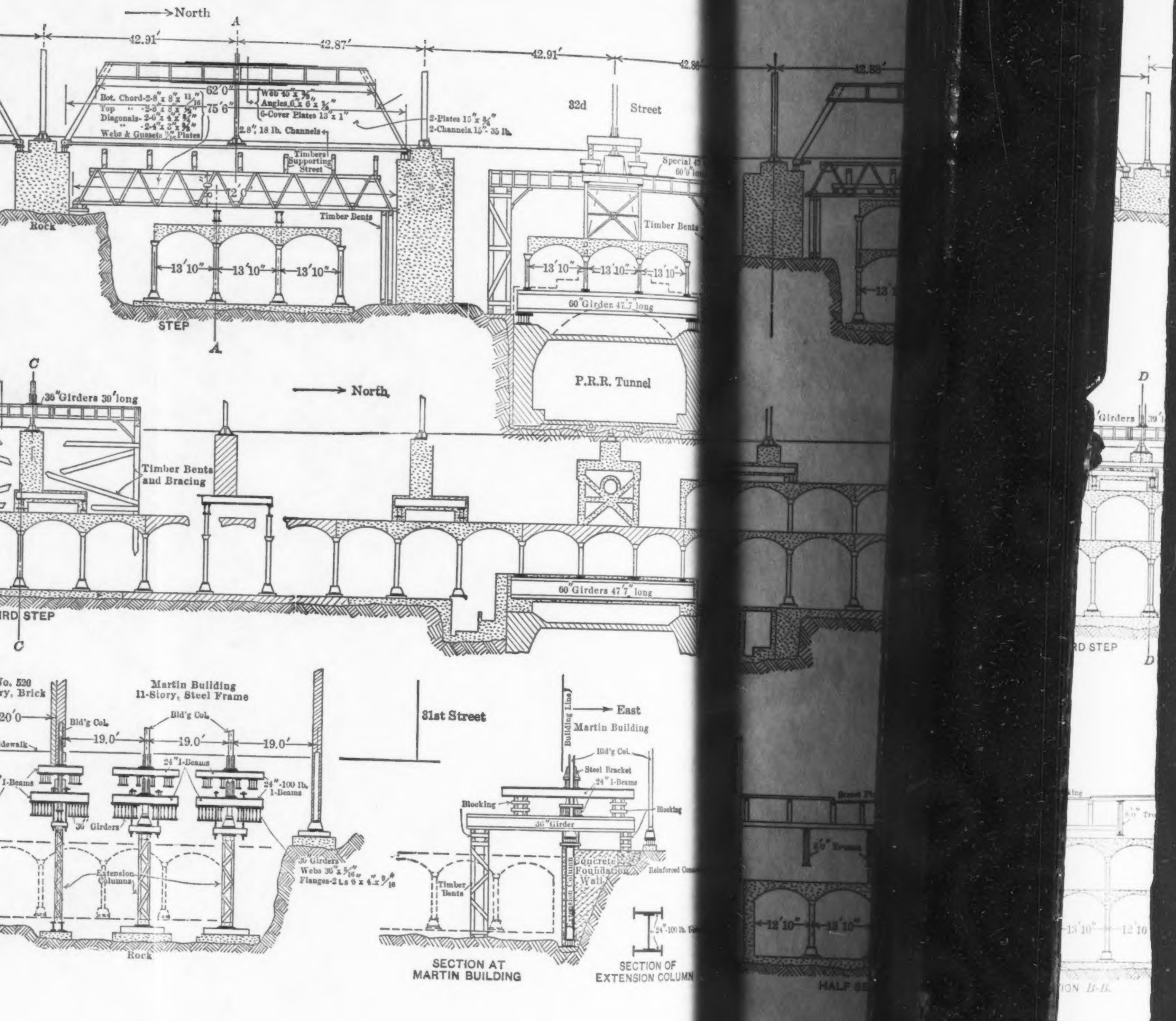


SECTION AT BANK BUILDING

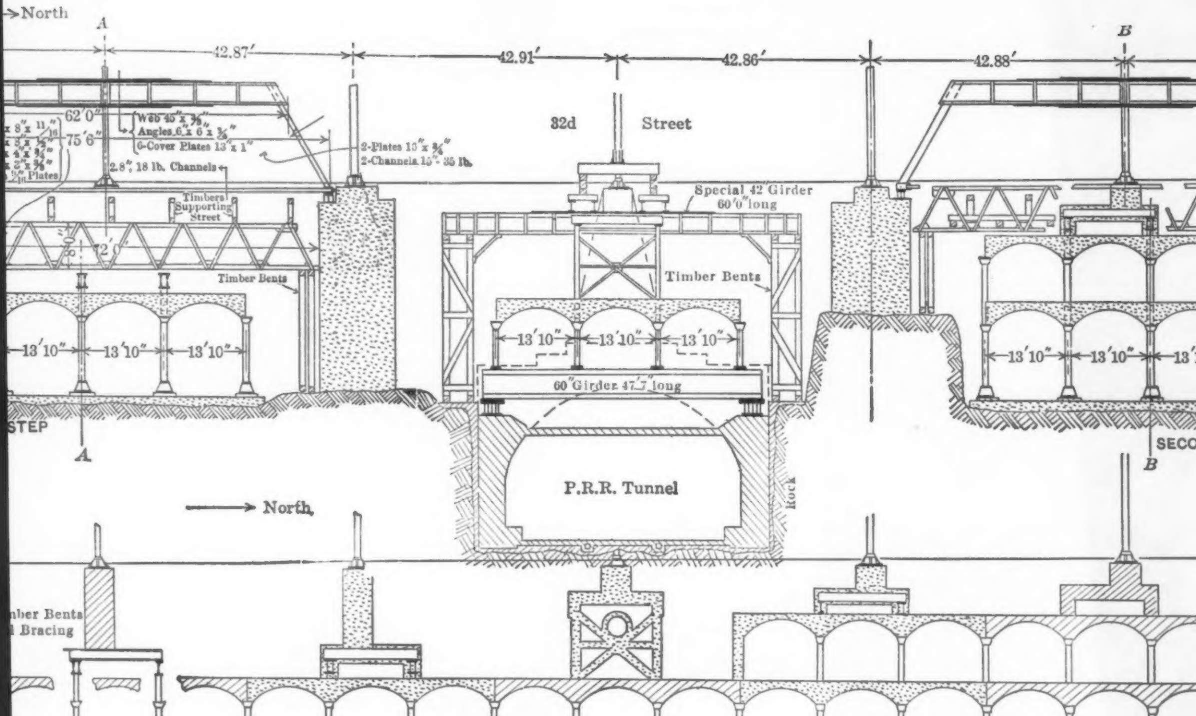


North ←

UNDERPINNING BUILDINGS
31st to 32d Street
Looking East.



North
A
42.91′
42.87′
42.91′
Bot. Chord-2-8″ x 8″ x 11/16″
Top
Diagonals-
Webs & Gussets Plates
62′0″
75′6″
Web
Angles
6-Cover Plates 13″ x 1″
2-Plates
2-Channels
18 lb. Channels
32d
Street
Timbers Supporting Street
Timber Bents
Rock
13′10″
STEP
60″ Girder
P.R.R. Tunnel
C
36″ Girders 30′ long
North
Timber Bents and Bracing
60″ Girders 47′7″ long
RD STEP
Martin Building
11-Story, Steel Frame
Bld'g Col.
19.0′
20′0
24″ I-Beams
24″-100 lb. I-Beams
36″ Girders
Extension Columns
36″ Girders
Rock
31st Street
Building Line
East
Martin Building
Bld'g Col.
Steel Bracket
24″ I-Beams
Blocking
36″ Girder
Timber Bents
Concrete Foundation Wall
Reinforced Concrete
SECTION AT MARTIN BUILDING
SECTION OF EXTENSION COLUMN
D
Girders
RD STEP
12′10″
HALF SE
SECTION B-B.

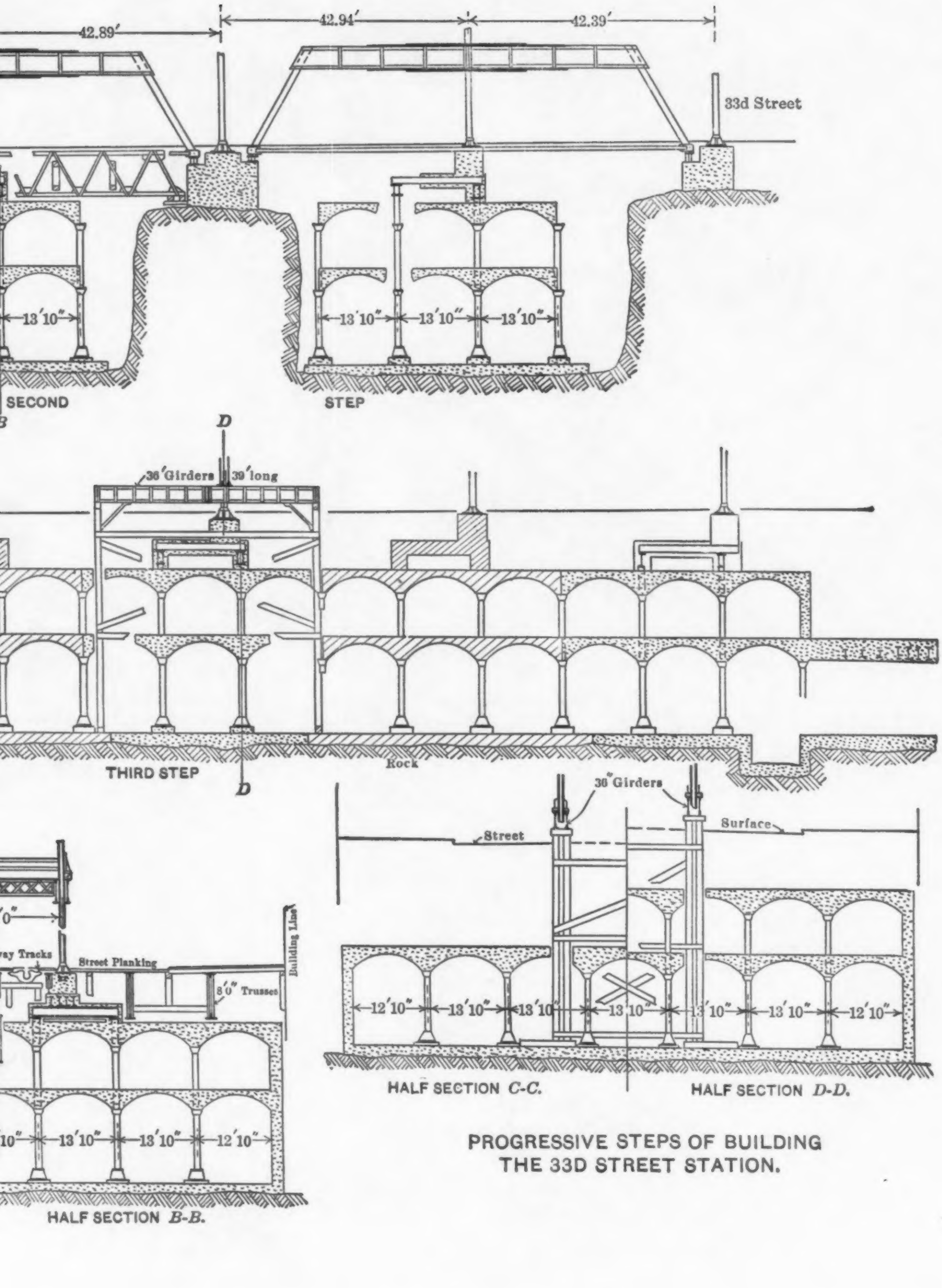


SECTION AT MARTIN BUILDING

SECTION OF EXTENSION COLUMN

HALF SECTION *A-A.*

PLATE LXV.
PAPERS, AM. SOC. C. E.
AUGUST, 1912.
BURROWES ON
SIXTH AVENUE SUBWAY,
H & M. R. R.



PROGRESSIVE STEPS OF BUILDING
THE 33D STREET STATION.

The elimination of dockage cost and rentals;
The reliable delivery of the materials in large quantities by scows as compared with trucking in small quantities.

The greater haul from the 42d Street yard involved a small additional cost over that of the 17th Street yard, but this was more than compensated by the loading of the materials directly from scows into trucks, which, without further delay, could then haul directly to the work.

Besides the storage and loading of materials, the following labor was performed at the yard:

The repair and assembly of plant,
The bending of reinforcing rods to shape,
The building of special forms for the concrete work,
The repairs and assembly of special shapes, etc.

Plant.

The plant was composed of the following main items:

1 power-house equipped with two Ingersoll-Rand compressors having a total capacity of 9 000 cu. ft. of free air per min.;
2 derricks having 25-h.p., Lidgerwood, double-drum engines, operated by compressed air;
8 derricks with Lambert and Lidgerwood 25 and 30-h.p., double-drum, electric engines;
2 cableways with 25-h.p., Lidgerwood, double-drum engines, operated by compressed air;
2 cableways with Lambert 25-h.p., double-drum, electric engines;
8 concrete mixers—¾-yd., Ransome—operated by compressed air;
26 pumps—Cameron and Worthington—operated by compressed air;
13 electric pumps—centrifugal;
40 rock drills, 2¼ and 3-in., operated by compressed air.

The cableways spanned one city block, or from 240 to 260 ft. between towers, and the derricks were all stiff-legged, with masts and booms of such sizes as to suit the conditions at the points of operation. All riveting hammers used for the steelwork were operated by compressed air. The power-house, supplying all the compressed air for the work, was in the yard at 42d Street and East River, and was that previously used for driving the Belmont Tunnel.

The curves on Figs. 1, 2, and 3 show the relative costs of the different items of the construction. The cost of plant is not included. The installation and operation are included in the different items of cost.

The value of the plant was estimated as follows:

Electric and compressed air hoist engines, new and old	$39 400
Concrete mixers	6 520
Pumps—new and old	4 980
Drills—new and old	6 550
Depreciation of power-house during use on this work	15 000
Frames and cables and supplies—new and old	7 000
Total estimated value of plant, as furnished on the work	$79 450

Other items usually considered as plant are included in the different cost curves.

The progress of the excavation, concrete work, etc., at various times is shown by the curves on Fig. 3.

The construction was carried on in two sections, the first (between 12th and 27th Streets) was completed before the commencement of the upper one, between 27th and 33d Streets.

The section between 12th and 27th Streets was commenced in September, 1906, and completed in September, 1909. The section from 27th to 33d Street was commenced in August, 1909, and completed in December, 1911.

The cost of the engineering on the work (exclusive of the main office, which is not included in the curves) was 2.78% of the construction.

Statistics.

The work was commenced in October, 1906, and was finished in November, 1911. In two portions of the work—between 12th and 24th Streets, and between 27th and 33d Streets, including the 33d Street Station—the time required from the commencement of the excavation to the operation of trains in the subway was 14 months, which is considered rapid construction. On the whole work there were only 5 deaths due to accident of any character.

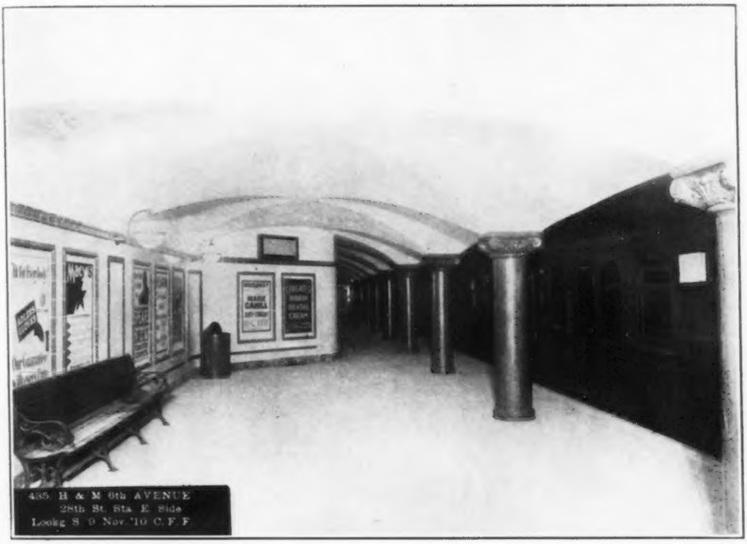
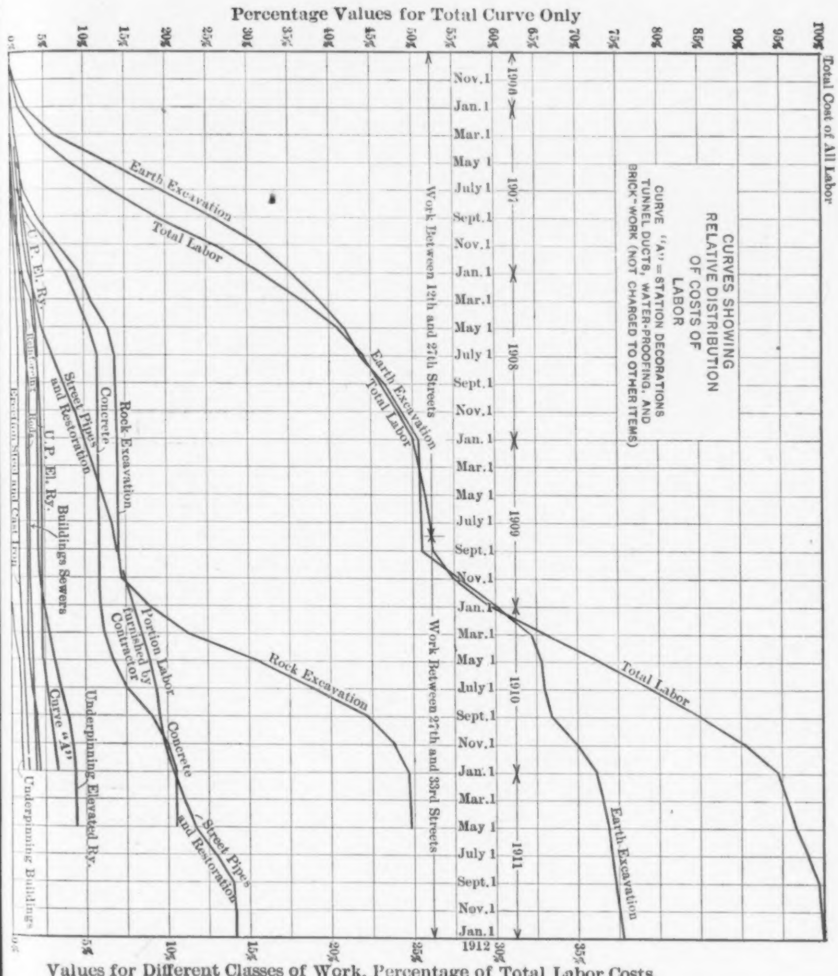


FIG. 1.—FINISHED PLATFORM IN STANDARD STATION.

FIG. 2.—COMPLETED PLATFORMS AND TRACK IN THIRTY-THIRD STREET STATION.


Underpinning Buildings


Percentage Values for Total Curve Only
Total Cost of All Labor
CURVES SHOWING RELATIVE DISTRIBUTION OF COSTS OF LABOR
CURVE "A" = STATION DECORATIONS TUNNEL DUCTS, WATER-PROOFING, AND BRICK WORK (NOT CHARGED TO OTHER ITEMS)
Work Between 12th and 27th Streets
Work Between 27th and 33rd Streets
Earth Excavation
Total Labor
Rock Excavation
Concrete
Street Pipes and Restoration
U. P. El. Ry.
Buildings
Sewers
Portion Labor furnished by Contractor
Curve "A"
Underpinning Elevated Ry.
Underpinning Buildings
Erection Steel and Cast Iron
1906
1907
1908
1909
1910
1911
1912
Values for Different Classes of Work. Percentage of Total Labor Costs.

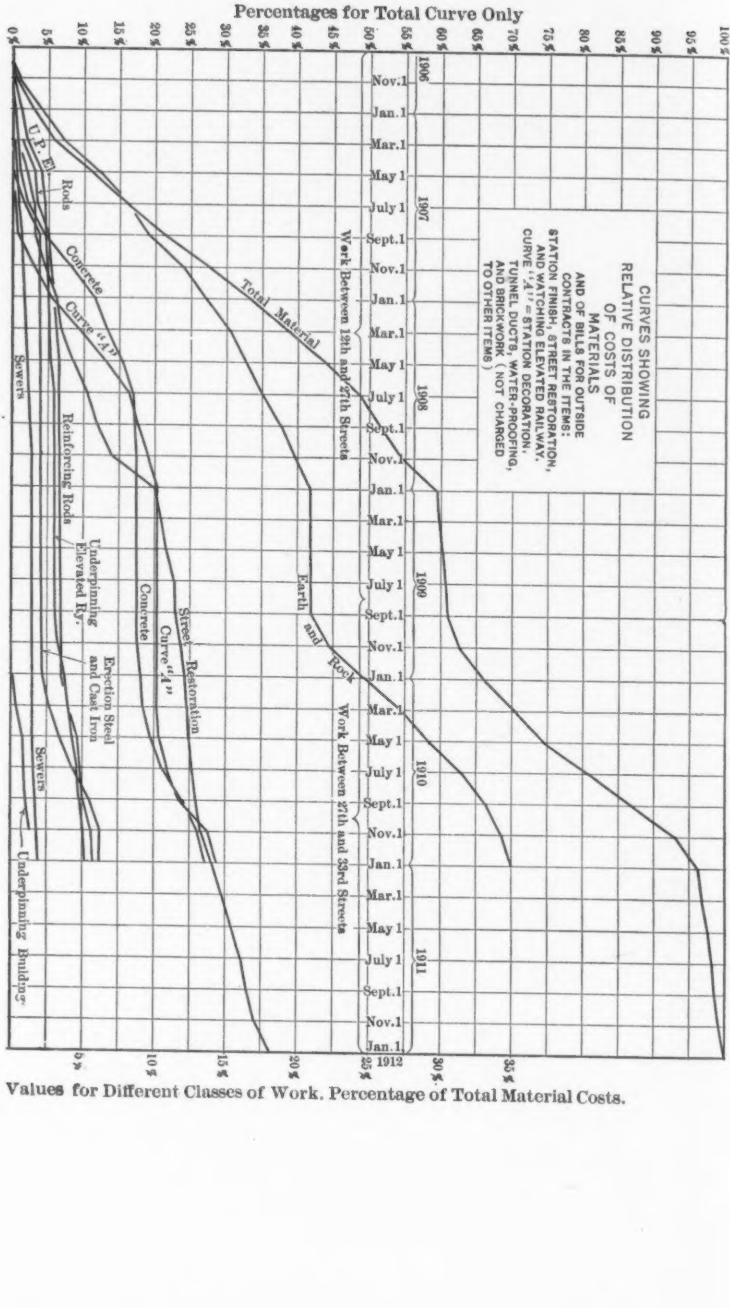

Percentages for Total Curve Only
CURVES SHOWING RELATIVE DISTRIBUTION OF COSTS OF MATERIALS AND OF BILLS FOR OUTSIDE CONTRACTS IN THE ITEMS: STATION FINISH, STREET RESTORATION, AND WATCHING ELEVATED RAILWAY. CURVE "A" = STATION DECORATION. TUNNEL DUCTS, WATER-PROOFING, AND BRICKWORK (NOT CHARGED TO OTHER ITEMS)
Total Cost of All Materials
Total Material
Earth and Rock
Work Between 12th and 27th Streets
Work Between 27th and 33rd Streets
Concrete
Curve "A"
Street Restoration
U.P. El.
Rods
Sewers
Reinforcing Rods
Underpinning Elevated Ry.
Erection Steel and Cast Iron
Underpinning Building
Values for Different Classes of Work. Percentage of Total Material Costs.

FIG. 2.

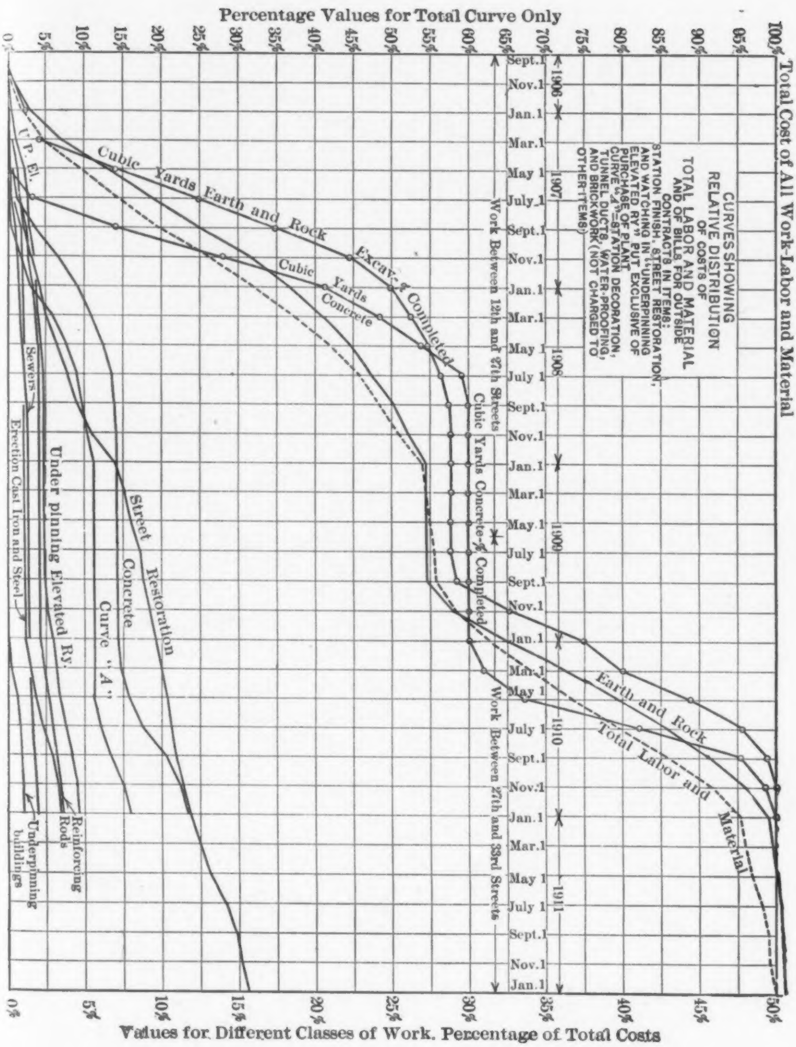

Percentage Values for Total Curve Only
Total Cost of All Work-Labor and Material
CURVES SHOWING RELATIVE DISTRIBUTION OF COSTS OF TOTAL LABOR AND MATERIAL AND OF BILLS FOR OUTSIDE CONTRACTS IN ITEMS: STATION FINISH, STREET RESTORATION, AND WATCHING IN "UNDERPINNING ELEVATED RY." PUT EXCLUSIVE OF PURCHASE OF PLANT. CURVE "A"=STATION DECORATION, TUNNEL DUCTS, WATER-PROOFING, AND BRICKWORK (NOT CHARGED TO OTHER ITEMS)
Work Between 12th and 27th Streets
Work Between 27th and 33rd Streets
Cubic Yards Earth and Rock Excav.-% Completed
Cubic Yards Concrete
Cubic Yards Concrete-% Completed
Earth and Rock
Total Labor and Material
U. P. El.
Sewers
Erection Cast Iron and Steel
Under pinning Elevated Ry.
Street Restoration
Concrete
Curve "A"
Reinforcing Rods
Underpinning buildings
1906
1907
1908
1909
1910
1911
Values for Different Classes of Work. Percentage of Total Costs

FIG. 8.

The total quantities of the different kinds of work were:

Earth excavation	220 900	cu. yd.
Rock "	95 350	" "
Concrete	65 180	" "
Reinforcement rods	7 090 000	lb.
Elevated railway underpinned	277	columns.
Sewers of different sizes (exclusive of the 18th Street main outlet sewer)	9 500	lin. ft.
Electric ducts in subway	278 500	duct feet.
Miscellaneous brickwork, not included in other items	5 200	cu. yd.
Structural steel and cast iron	2 834	tons.
Interior finish of stations	5	stations.
Street restored	5 400	lin. ft.
Buildings (from 4 to 11 stories) underpinned	340	lin. ft.
Field engineering	5	years.

The loadings of the street were estimated to be:

Surface railway	$7\frac{1}{2}$	tons per lin ft. of street.
Roadways	$6\frac{1}{2}$	" " " " " "
Sidewalks	3	" " " " " "
Total	17	tons per lin. ft. of street.

The total quantity of timber used in the excavation, including the planking for the street surface, was 7 703 000 ft. b. m., or 24.36 ft. b. m. per cu. yd. of excavation of both earth and rock, unclassified.

Between July, 1909, and August, 1910, there were 30 700 drill holes blasted, each hole requiring an average of 1.20 lb. of dynamite. The average quantity of dynamite for all rock excavated from 12th to 33d Streets was 0.58 lb. per cu. yd.

The proportions of labor and material for each kind of work was as follows:

	Percentage for material.	Percentage for labor.
In earth excavation	67.78	32.22
In rock excavation	72.95	27.05
Concrete	49.70	50.30
Reinforcing rods	29.98	70.02
Underpinning elevated railway	51.64	48.36
Sewer construction	57.81	42.19
Structural steel and cast iron	21.36	78.64

PLATE LXVII.
PAPERS, AM. SOC. C. E.
AUGUST, 1912.
BURROWES ON
THE SIXTH AVENUE SUBWAY,
H. & M. R. R.

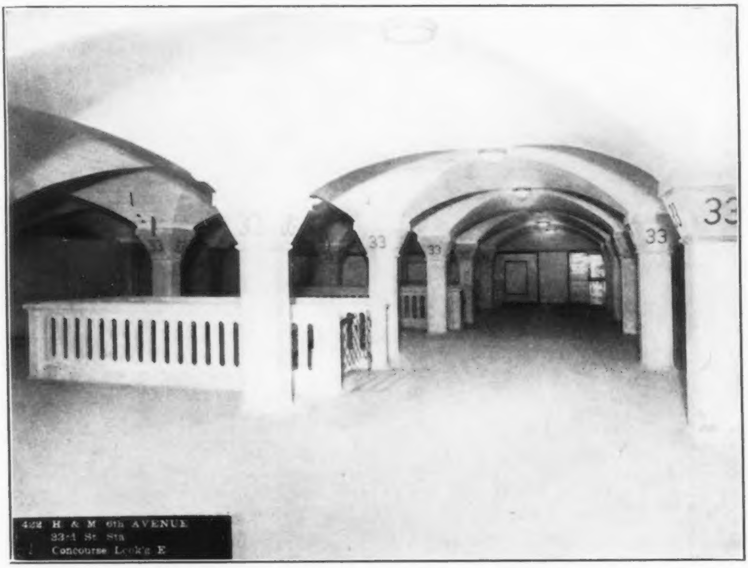


FIG. 1.—VIEW ACROSS CONCOURSE FLOOR OF THIRTY-THIRD STREET STATION.

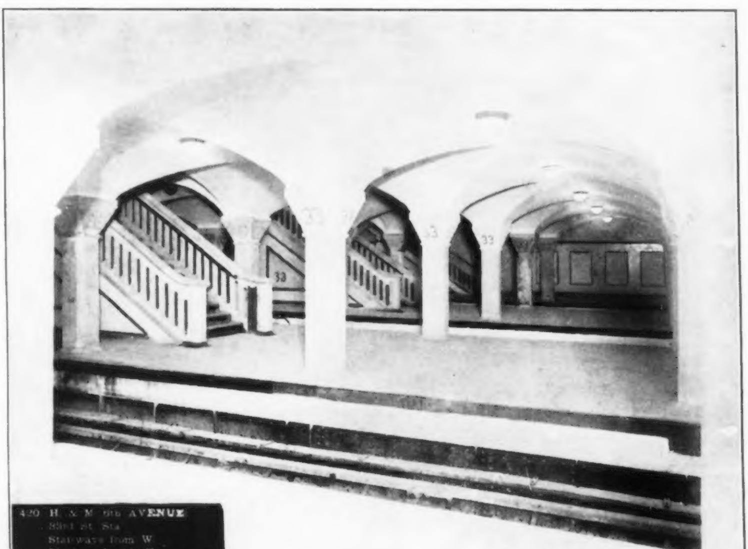


FIG. 2.—VIEW ACROSS THIRTY-THIRD STREET STATION SHOWING TRAIN PLATFORMS.

In all the other items of construction there were included in the "Materials," bills for both labor and materials, and the relative proportions for labor and materials were not ascertained.

The average depth of the holes drilled in rock was 14 ft., and the average depth of hole per drill per day was 35 ft.

The proportion of reinforcement rods (in that portion of the concrete which was reinforced) was 106 lb. per cu. yd. of concrete.

The proportions of the different items of cost of construction were as follows:

	Percentage of total cost.
Earth excavation	31.32
Rock excavation	19.16
Concrete (exclusive of rods)	11.74
Reinforcement rods	3.50
Underpinning elevated railway	4.55
Sewer construction	1.69
Tunnel ducts (electric)	0.52
Brickwork (portion not included in other items)	1.35
Structural steel and cast iron	3.34
Station, interior finish	3.29
Street restoration (including relaying pipes, etc)	15.66
Water-proofing	1.23
Underpinning buildings	0.98
Four entrances through private property	1.67
	100.00

Charles M. Jacobs, M. Am. Soc. C. E., was Chief Engineer and J. V. Davies, M. Am. Soc. C. E., was Deputy Chief Engineer until August 1st, 1909, when Mr. Jacobs became Consulting Chief Engineer, Mr. Davies, Chief Engineer, and James Forgie, M. Am. Soc. C. E., Deputy Chief Engineer. G. D. Snyder, M. Am. Soc. C. E., was Principal Assistant Engineer. The writer was Resident Engineer in direct charge of the work, and Mr. O. J. Theiss and Leslie Muller, Assoc. M. Am. Soc. C. E., were Assistant Engineers in charge of the work in the field. Mr. William Langston was Chief Inspector.

The Degnon Contracting Company were the contractors for whom H. C. Sanford, Assoc. M. Am. Soc. C. E., was Chief Engineer and R. P. Gustin, Assoc. M. Am. Soc. C. E., was Engineer in Charge. Mr. M. J. Morris was Superintendent.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

THE STRENGTH OF COLUMNS.*

By W. E. Lilly, Esq.†

Of recent years many data have been accumulated from the published results of tests of the strength of columns and struts. The application of these tests to the various cases that occur in the design of columns, however, gives rise to some difficulties, owing principally to a want of some general method by which the experimental data may be co-ordinated. For instance, from the individual tests on braced struts, the proportions to be adopted for bracing can be approximately determined, but no rules based on these tests have been formulated which are generally applicable.

In this paper it is proposed to consider briefly the derivation of the Rankine-Gordon formula and the writer's formula for columns, and then compare some of the values obtained from tests on columns with those derived from the formulas applicable to the particular cases considered. Some criticism on column formulas and their application to the design of struts will then be given, and some suggestions offered as to the lines on which the co-ordination of experimental work in this direction should proceed.

A bibliography of the literature of the subject is given in the appendix, together with titles of papers by the writer referred to in the text.

At different times various formulas have been put forward to determine the strength of columns; of these, the writer considers the Rankine-Gordon formula to be the best. The following quotation

*This paper will not be presented at any meeting, but written communications on the subject are invited for publication with it in *Transactions*.

† Lecturer in Engineering, University of Dublin.

from Mansfield Merriman,* M. Am. Soc. C. E., also supports the writer's opinion:

"The theoretic basis of Rankine's formula seems far more satisfactory than that of any other which has been proposed for the discussion of such columns as are used in engineering practice."

This formula is of a form proposed by Tredgold and afterward revised by Gordon, who determined the approximate value of the constants; it was then modified by Rankine, who substituted the value of the radius of gyration for the diameter of the cross-section. The formula only applies to solid columns, and requires modification when used for hollow or irregular forms of cross-section. It may be derived as follows: Consider the column of uniform cross-section shown by Fig. 1, and assume that it is slightly bent by the load, P; then the bending moment at the middle is

$$M = P\ d = p\ A\ d \qquad (1)$$

Where P = the load on the column,
d = the central deflection,
p = the load per unit area,
and A = the area of the cross-section.

From the theory of flexure,

$$\frac{M}{I} = \frac{p_1}{y} \qquad (2)$$

Where I = the moment of inertia of the cross-section about a diameter,
y = the distance of the outer fiber from the neutral axis,
and p_1 = the intensity of the outer fiber stress due to bending.

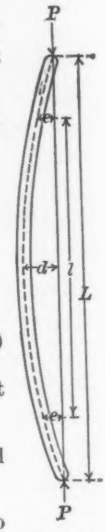


Fig. 1.

From Equations 1 and 2 the following relation obtains:

$$p\ A\ d = \frac{p_1\ I}{y}; \text{ or } \frac{p_1}{p} = \frac{dy}{\rho^2} \qquad (3)$$

since $I = A\ \rho^2$, where ρ is the radius of gyration of the cross-section.

If the cross-section of the column is considered as being made up of two areas, as shown by Fig. 2, one of which, A_1, transmits the direct stress and the other, A_2, transmits the bending stress; then, if the outer

* "Mechanics of Materials."

fiber stress due to bending is equal to the direct stress, $P = p\ A$ may be put equal to $p_1\ A_1$; also, $I = A_2 \rho^2$, giving

$$\frac{p_1}{p} = \frac{A_2}{A_1} = \frac{dy}{\rho^2} \quad \text{.....................} \quad (4)$$

This relation obtains because any alteration of the breadth of a given cross-section does not alter the value of ρ. Adding unity to both sides of Equation 3 and putting $p + p_1 = f$, the strength to compression of the material, gives

$$p = \frac{f}{1 + \frac{dy}{\rho^2}} \quad \text{.....................} \quad (5)$$

In applying Equation 5 to columns and struts, the difficulty arises as to the determination of what function of the length applies to d, and it can only be approximated. The Rankine-Gordon formula assumes that the area, A_1, transmits only the direct stress and is not affected by the bending stress, and that the area, A_2, transmits the bending stress and is not affected by the direct stress; and further, that, when the column is about to break, the intensity of the direct stress on A_1 is then equal to the intensity of the bending stress on the outer fiber of A_2 This assumption gives the first approximation derived from theory, and errs on the side of safety, because the strength of the column, considered as a whole, is greater than that of the assumed compound column.

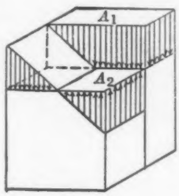


FIG. 2.

From the theory of flexure, the deflection of the column is given by the expression

$$d = \frac{f\ L^2}{\pi^2\ E\ y} \quad \text{......................} \quad (6)$$

where E = Young's modulus of elasticity, L = the length of the column, and the other terms have the signification already given. This expression is the same as that obtained for a uniform beam supported at its extremities and carrying a load proportional to the ordinates of a sine curve; the deflection curve is then also a sine curve. Substituting for d in Equation 5 from Equation 6 gives

$$p = \frac{P}{A} = \frac{f}{1 + \frac{f\ L^2}{\pi^2\ E\ \rho^2}} \quad \text{.................} \quad (7)$$

which is the form of the Rankine-Gordon formula for centrally loaded, round-ended columns. In order to modify it to include eccentrically loaded columns, assume that the sinusoidal deflection curve closely approximates to the parabola; then, from Fig. 1,

$$d = e + \frac{f\,l^2}{\pi^2 E \rho^2} \qquad (8)$$

where e = the eccentricity of loading.

Substituting for d in Equation 5 from Equation 8 gives

$$p = \frac{P}{A} = \frac{f}{1 + \frac{e\,y}{\rho^2} + \frac{f\,l^2}{\pi^2 E \rho^2}} \qquad (9)$$

which is the modified form of the Rankine-Gordon formula for eccentrically loaded columns.

The writer has given the further modification of the formula to include transversely loaded columns which can be derived as follows: From Equations 9 and 4,

$$\frac{A_1}{A_2}\left(\frac{e\,y}{\rho^2} + \frac{f\,l^2}{\pi^2 E \rho^2}\right) = 1$$

but

$$\frac{P\,e}{I} = \frac{f\,A_1\,e}{A_2\,\rho^2} = \frac{p_o}{y}$$

hence

$$\frac{p_o}{f} + \frac{f\,l^2}{\pi^2 E \rho^2}\,\frac{A_1}{A_2} = 1$$

or

$$\frac{f}{f - p_o}\,\frac{f\,l^2}{\pi^2 E \rho^2} = \frac{A_2}{A_1} \qquad (10)$$

giving, from Equation 4,

$$p = \frac{P}{A} = \frac{f}{1 + \frac{f\,l^2}{\pi^2 E \rho^2}\,\frac{f}{f - p_o}} \qquad (11)$$

To apply the formula given in Equation 10, a supplementary calculation is necessary in order to determine the load required to produce a given fiber stress, p_o. Writing Equation 10 in the form,

$$A_2 = \frac{A}{1 + \frac{\pi^2 E \rho^2}{f\,l^2}\,\frac{f - p_o}{f}} \qquad (12)$$

Now

$$\frac{p_o}{y} = \frac{M}{A_2\,\rho^2}, \text{ or } A_2 = \frac{M\,y}{p_o\,\rho^2},$$

and substituting for A_2 in Equation 12 gives

$$M = \frac{p_o \rho^2}{y} \frac{A}{1 + \frac{\pi^2 E \rho^2}{f l^2} \frac{f - p_o}{f}} \ldots\ldots\ldots\ldots\ldots (13)$$

To determine M, the kind of normal loading must be specified; for a uniform load, $M = \frac{w l^2}{8}$, where w is the intensity of the load per unit length on the column; hence, for a given column, when M is given, p_o can be determined from Equation 13, and conversely.

For the uniform load, the shear stress, S, is $\frac{w l}{2}$, and this is taken up by the supports at the ends of the column; there is also the internal shear stress of the column to be considered. Usually, in a beam or girder, it is the shearing stresses which give rise to the bending moments; in columns, however, it is the variations of the bending moments which give rise to the shearing stresses. If, then, a column is bent to its proof deflection, so also should the web (if properly designed) come to its proof stress. Now consider the column as a beam supported at its extremities and bent to a sinusoidal curve. If w is the intensity of the load per unit area, and d is the central deflection, the bending moment

$$M = \frac{w d l^2}{\pi^2}, \text{ and } S = \frac{w d l}{\pi}$$

where S is the shearing force at one support. Hence,

$$S \frac{l}{\pi} = M = \frac{f I}{y} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (14)$$

Now $M = Pd$ for the column, and substituting for P and d from Equations 6 and 7 gives

$$S \frac{l}{\pi} = \frac{f A}{1 + \frac{f l^2}{\pi^2 E \rho^2}} \frac{f l^2}{\pi^2 E y}$$

$$\text{or } S = \frac{f I}{y} \frac{1}{\frac{l}{\pi} + \frac{\pi E \rho^2}{f l}} \ldots\ldots\ldots\ldots\ldots\ldots (15)$$

From this equation the thickness of the web for box-columns can be determined; for braced columns, the area of the cross-section of the bracing is given by the equation, $a = \frac{S}{f}$ sec. θ, where θ is the angle of the bracing with the direction of the shearing force. It follows,

then, from Equations 13 and 15, that in designing columns the external and internal shear stresses must be considered together.

The formulas given in Equations 7, 9, and 11 can be written in the generalized form

$$p = \frac{P}{A} = \frac{f}{1 + \frac{f}{f - p_o}\left(\frac{l\,y}{\rho^2} + \frac{f\,l^2}{\pi^2\,E\,\rho^2}\right)} \quad \text{.......... (16)}$$

which may be regarded as the modified form of the Rankine-Gordon formula extended to include eccentric and transversely loaded columns. The formula only applies to solid columns, and requires modification when applied to columns of irregular or hollow forms of cross-section.

If a short, hollow, cylindrical tube be tested under direct compression in the testing machine, it fails by secondary flexure or wrinkling, and the tube breaks up into a series of waves. From a comparison of the results obtained from a large number of tests by the writer, it was shown that, as the tube becomes larger in diameter and the thickness less, the load producing failure becomes smaller, and it is not until the length of the tube is less than a wave length that the load producing failure approaches the resistance to compression of the material. Hence the true strength to compression of the tube is the load which produces the wave formation. From the experiments, and also from the analysis, it was found that the wave length varied as the square root of the area of the cross-section. This result leads to the following equation for the limiting load:

$$f_c = \frac{f}{1 + k\,\frac{\rho}{t}} \quad \text{...................... (17)}$$

where f_c = the limiting load per unit of area on a column of one wave length,

f = the strength to compression of the material,

k = a constant,

ρ = the radius of gyration of the circular cross-section about a diameter,

t = the thickness of the circular cross-section.

Further experiments on columns of square, triangular, and H sections showed that the above formula applied, not only to circular sections, but to these sections as well, the coefficient, k, having a particular value for each cross-section.

Referring now to the Rankine-Gordon formula given in Equation 7: The value of f, the strength to compression, was assumed as constant, whereas the experiments show that it is a variable which involves the phenomena of secondary flexure, and therefore should be replaced by the value, f_c, given in Equation 17. The following formula is then obtained:

$$p = \frac{P}{A} = \frac{f}{1 + k\,\frac{\rho}{t} + \frac{f\,l^2}{\pi^2\,E\,\rho^2}} \dots\dots\dots\dots\dots\dots (18)$$

This formula takes into consideration both primary and secondary flexure, and its solution involves the length and the figure and thickness of the cross-section of the column. Also, by giving values to k pertaining to the different figures of the cross-section, it can be applied generally to all forms of struts.

When eccentric and transverse loading require to be taken into consideration, the formula, by substitution from Equation 17 in Equation 16, becomes

$$p = \frac{P}{A} = \frac{f}{\left(1 + k\,\frac{\rho}{t}\right)\left(1 + \frac{e\,y}{\rho^2}\,\frac{f}{f - p_o}\right) + \frac{f\,l^2}{\pi^2\,E\,\rho^2}\,\frac{f}{f - p_o}} \dots (19)$$

Table 1 gives the constants to be used in the foregoing formulas for different materials; these have been deduced from a critical examination of the tests on columns and from the writer's experiments. The value of the constant, c, is $\frac{f}{\pi^2\,E}$ for round-ended columns and $\frac{f}{4\,\pi^2\,E}$ for fixed-ended columns.

TABLE 1.

Material.	Strength to compression, f, in pounds per square inch.	Modulus of elasticity, E.	Round-ended columns, c.	Fixed-ended columns.
Nickel steel.....	120 000	31 000 000	$\frac{1}{2\,800}$	$\frac{1}{11\,200}$
Bessemer steel............	110 000	31 000 000	$\frac{1}{2\,600}$	$\frac{1}{10\,400}$
Mild steel..................	80 000	30 000 000	$\frac{1}{4\,000}$	$\frac{1}{16\,000}$
Mild steel, annealed.......	60 000	30 000 000	$\frac{1}{5\,000}$	$\frac{1}{20\,000}$
Wrought iron, annealed...	55 000	26 000 000	$\frac{1}{4\,500}$	$\frac{1}{18\,000}$
Cast iron..................	110 000	14 000 000	$\frac{1}{1\,300}$	$\frac{1}{5\,200}$

The coefficient, k, is of the form $\frac{N f}{E}$, where N represents the tabular number of the figure of the cross-section of the column. On Fig. 3 are shown the numbers obtained for a few types of cross-section; these have been obtained from tests on small specimens, for which reason they can only be considered as approximate.

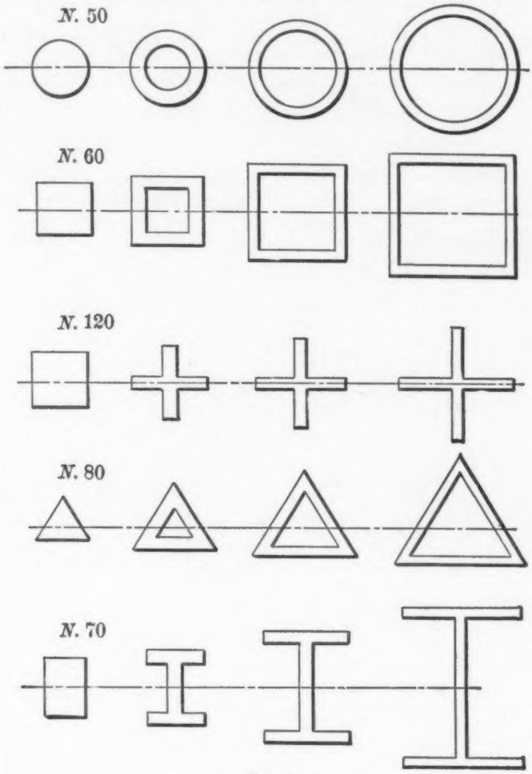


FIG. 3.

On Fig. 4 are shown the results of some tests by the writer on mild-steel columns, ½ in. in diameter. The values obtained for the solid, round-ended columns under central loading are shown by Curve *A*, and the plotted values from the Rankine-Gordon formula by Curve *2*. It will be noticed that these curves intersect at the value of $\frac{l}{\rho}$ = about 40 to 45. For values of $\frac{l}{\rho}$ less than 45 the values given by the formula

are slightly higher than the experimental ones, and for values of $\frac{l}{\rho}$ between 45 and 120 those given by the formula are somewhat less than those obtained from the experiments. For values of $\frac{l}{\rho}$ greater than 120 the calculated and experimental values sensibly agree with Curve *1*, obtained from Euler's formula. The writer has shown that the experimental curves can be closely approximated by assuming the deflection curve of the column to vary in some proportion less than the square of the length; the resulting formulas then become more complex and are of little use for the practical design of columns. For this reason the writer adheres to the Rankine-Gordon as a practical working

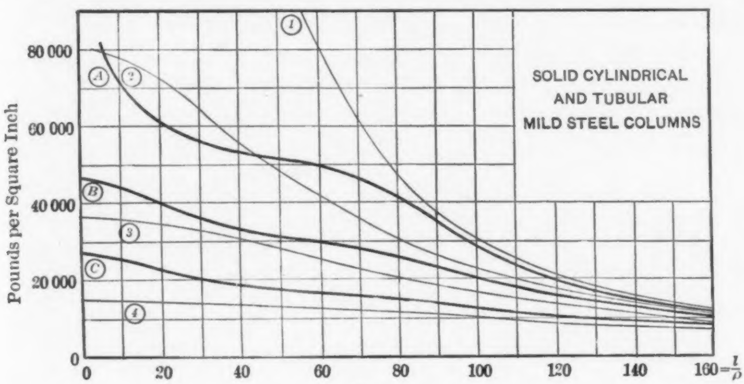


FIG. 4.

formula. The values obtained by its use, except for very short columns, always err on the side of safety, and the simplicity of its form must always recommend it to engineers. When evaluating the constants for the formula from experimental data, it is necessary to bear in mind the relations between the experimental and calculated curves, as just pointed out. The same remarks apply, but in a much smaller degree, when allowing for secondary flexure in the writer's formulas. Curves *B* and *3* show this. Curve *B* relates to a series of tests on centrally loaded, mild-steel tubes, $\frac{3}{4}$ in. in diameter and 18 S.W.G. thick, these have been selected to show the influence of secondary flexure on the strength of such columns. Curve *3* shows the calculated values from the formula. Curves *C* and *4* relate to the same tubes but having an eccentricity of loading of $\frac{1}{4}$ in. In these experiments there is little

variation in the loads producing failure for a large range of the length, owing to the small deflection of the column under the applied load compared with the eccentricity of loading.

The values obtained by the use of the foregoing formulas will now be compared with the results obtained from some experimental tests on columns.*

The first results are those of compression tests on channel bars of latticed columns.† From the writer's experiments, the value of f, for solid, rolled, wrought-iron bars, is about 60 000 to 65 000 lb. per sq. in. Assuming the greater value, then Equation 17 becomes

$$f_c = \frac{65\,000}{1 + k\frac{\rho}{t}} \qquad \text{(20)}$$

From the sketches‡ of the different sections, it is only possible to approximate to the values of $\frac{\rho}{t}$ of the channel sections. Taking the value of ρ about an axis perpendicular to the long side of the section, and dividing by the thickness, gives $\frac{\rho}{t}$ approximately 9, 10, 11, and 12, for the 6, 8, 10, and 12-in. sections, respectively. These numbers give $k = 0.06$, approximately, and Equation 20 becomes

$$f_c = \frac{65\,000}{1 + 0.06\frac{\rho}{t}} \qquad \text{(21)}$$

and the values of f_c for the 6, 8, 10, and 12-in. sections are 42 000, 40 500, 39 000, and 37 800 lb. per sq. in., respectively. On comparing these numbers with those given from the tests on the shorter lengths of the channel bars they will be found to be in close agreement.

Substituting from Equation 21 in the Rankine-Gordon formula gives

$$p = \frac{P}{A} = \frac{65\,000}{1 + 0.06\frac{\rho}{t} + \frac{1}{4\,300}\frac{l^2}{\rho^2}}$$

E being taken at 28 000 000. The data in the sketches and tables are not sufficiently precise to allow of any great accuracy in determining

*Some of the tables showing the results of these tests are contained in Professor Lanza's "Applied Mechanics" (Ninth edition), and some are in the *Transactions* of this Society.

†Lanza, "Applied Mechanics," pp. 422-423.

‡Lanza, "Applied Mechanics," pp. 427-428.

the values of ρ, of which two are to be considered, one in a plane parallel to the lacing and the other in a plane perpendicular thereto. Using the smaller values in the formula gives numerical results which approximate to the experimental values in the tables.* The constant, $\frac{1}{4\,300}$, has been deduced for round-ended columns. For the tests given in the tables the friction of the pins prevents this condition from being satisfied, and therefore the constant requires to be modified for these tests; a value of about $\frac{1}{5\,000}$ should be used.

The manner of failure, as stated in the tables, gives but little information as to the way in which the lacing was affected when failure of the column took place. As already pointed out, in a laced column the proportions of the lacing depend on the shear stresses due to primary flexure of the column. Reverting to Equation 15, and applying it to the test column, No. 463,† gives the thrust in the lacing as about 1 750 lb. For this column, the following are the approximate data: From Equation 21, $f = 42\,000$ lb. per sq. in., $A = 4.68$, $\rho = 3$ in. (nearly), $l = 240$ in., and $y = 4.5$. Substituting these values in Equation 15 gives

$$S = \frac{42\,000 \times 4.68}{4.5} \times \frac{3}{\frac{80}{\pi} + \frac{670\,\pi}{80}} = 2\,500 \text{ lb.}$$

The lacing slopes at 45° (nearly), giving

$$P = S \sec. \theta = 2\,500 \times \sqrt{2} = 3\,500 \text{ lb. (nearly).}$$

This thrust is taken up on two bars of the lacing, giving 1 750 lb. on each.

The size adopted for the lacing was 1.95 by ¼ in. in cross-section and 11.3 in. long between rivet centers. Assuming the bar to have been hinged at the ends, the safe thrust it would carry, by Equation 7, would be

$$P = \frac{65\,000 \times 1.95 \times \frac{1}{4}}{1 + \frac{24\,500}{4\,300}} = 4\,850 \text{ lb., approximately.}$$

As the calculated thrust was only 1 750 lb., the bracing was of ample strength. This seems to have been the case with nearly all the experi-

* Lanza, "Applied Mechanics," pp. 422, 423, 425.

† Lanza, "Applied Mechanics," p. 425.

ments given in these tables,* more especially as no particular failures of the lacing were recorded.

In addition to the shear stress due to primary flexure, there also arises the question of the proportions to be adopted for the lacing when secondary flexure is considered. If the lacing is made uniform throughout the column, and is of sufficient strength to transmit the shear stresses, it is also of ample strength to resist deformation due to secondary flexure. The writer has shown that, in the case of the usual box- and **H**-sections, the wave length of the wave formation is less than the breadth or width of the section. In a laced column, the channel bars are forced by the lacing to take up a wave length of twice the length of the lattice spacing, and this quantity is always greater than the size of the section. Under these conditions the stresses set up in the lacing due to secondary flexure are small compared with the shear stress at the ends of the column.

In a few tests of Phœnix columns,† the constants deduced therefrom, on substitution in Equation 18, give

$$p = \frac{P}{A} = \frac{70\,000}{1 + 0.04\,\frac{\rho}{t} + \frac{1}{4\,000}\,\frac{l^2}{\rho^2}}.$$

In using the formula, it is to be noted that the columns were tested with flat ends, and therefore the ends of the column were sensibly fixed. For these columns, the constant, $\frac{1}{4\,000}$, is multiplied by $\frac{1}{4}$. The coefficient, $k = 0.04$, for these sections, has the smallest value yet obtained from tests on columns. Therefore, they give the column of maximum strength for a given quantity of material. The writer has previously drawn attention to the advantage of stiffened and cellular forms of cross-sections for columns.

The results of a number of tests on wrought-iron pipe columns at the Massachusetts Institute of Technology‡ will be considered next. The deduced value of the constants, on substitution in Equation 18, gives

$$p = \frac{P}{A} = \frac{65\,000}{1 + \frac{1}{10}\,\frac{\rho}{t} + \frac{1}{4\,000}\,\frac{l^2}{\rho^2}}.$$

* Lanza, "Applied Mechanics," pp. 422, 423, 425.

† Lanza, "Applied Mechanics," p. 425.

‡ Lanza, "Applied Mechanics," p. 440.

This formula agrees closely with the writer's experiments, on tubes of much smaller diameters, in the laboratory of Trinity College, Dublin.

Now examine some tests on wrought-iron **Z**-bar columns by C. L. Strobel, M. Am. Soc. C. E.* The **Z**-irons used in making the columns were $2\frac{1}{2}$ by 3 by $2\frac{1}{2}$-in. and $\frac{5}{16}$ in. thick. The constants deduced from these tests, on substitution in Equation 18, give

$$p = \frac{P}{A} = \frac{65\,000}{1 + 0.4 + \frac{1}{4\,000}\frac{l^2}{\rho^2}}.$$

The columns were tested with fixed ends, and therefore the constant, $\frac{1}{4\,000}$, is to be multiplied by $\frac{1}{4}$. The value of $k\frac{l}{t}$ for the section of these columns cannot be given until further experimental data have been obtained.

In some tests on eight full-sized Bessemer steel bridge columns by the late J. G. Dagron, M. Am. Soc. C. E.,† the deduced value of the constants, on substitution in Equation 18, gives

$$p = \frac{P}{A} = \frac{80\,000}{1 + 0.5 + \frac{1}{4\,000}\frac{l^2}{\rho^2}}.$$

These were latticed columns, and were tested with pin ends. It is necessary to make a slight allowance for the friction of the pins, which increases the value of the constant from $\frac{1}{4\,000}$ to about $\frac{1}{6\,000}$. The lattice bars were $1\frac{3}{4}$ by $\frac{1}{4}$ by $14\frac{1}{2}$ in., center spaced, and 18 in. apart. Reverting to Equation 15, and substituting the data from the table, the thrust in the lattice bars at the ends is approximately 3 500 lb., and the breaking load on the lattice bars, calculated from Equation 7, gives 3 300 lb., showing that the proportions for the bracing were closely correct. The columns invariably failed by crippling, the lattice bars at the ends of the columns remaining intact.

The data, details, and results of some tests on mild-steel and wrought-iron columns, by Professors Talbot and Moore‡ will now be

* *Transactions*, Am. Soc. C. E., Vol. XVIII, 1888, p. 103; also Lanza, "Applied Mechanics," p. 439.

† *Transactions*, Am. Soc. C. E., Vol. XX, 1889, p. 254; also Lanza, "Applied Mechanics," p. 495.

‡ *Transactions*, Am. Soc. C. E., Vol. LXV, 1909, p. 202; also, *Bulletin No. 44*, University of Illinois, June, 1910.

examined. The deduced value of the constants from Test 17 on the mild-steel column, No. 1, on substitution in Equation 18, gives

$$p = \frac{P}{A} = \frac{60\,000}{1 + 1.4 + \frac{1}{4\,600}\frac{l^2}{\rho^2}}.$$

The tension tests show that the steel was of a very mild quality, and for this reason the value of $f = 60\,000$ has been adopted. The value of the coefficient, k, is approximately 0.09. The column failed by the sudden buckling of the flange between adjacent lattice bars, and therefore the ends of the column may be regarded as having been partly fixed. Reverting to Equation 15, and substituting the values from the data, the shear stress is determined as being approximately 3 800 lb. and the thrust in two lattice bars about 4 250 lb. The lattice bars were $1\frac{1}{4}$ by $\frac{7}{16}$ by 18 in., and the breaking load on these bars, calculated by Equation 5, gives 6 500 lb., and for two bars 13 000 lb., showing that they were of ample strength. It will be noted that the column failed by crippling. In Test No. 1* the lattice bars were 1 by $\frac{1}{4}$ in. The breaking load on these bars, calculated by Equation 7, gives 1 000 lb. (nearly), and 2 000 lb. for two bars, showing that they had only half the required strength. In this test the column failed by the buckling of the lacing.

The deduced value of the constants, from the tests on wrought-iron columns, on substitution in Equation 18, gives

$$p = \frac{P}{A} = \frac{55\,000}{1 + 0.6 + \frac{1}{5\,000}\frac{l^2}{\rho^2}}$$

and the value of the coefficient, k, approximately 0.08. These columns failed by crippling, no failure of the lacing being recorded. Calculating the value of the shear stress from Equation 15 shows that the lacing was of ample strength.

The object of this paper is to draw attention to the merits of the Rankine-Gordon formula, and of the writer's formula for co-ordinating the experimental data obtained from tests on columns. The formula takes into consideration eccentric and normal loading and secondary flexure, which covers most of the factors that arise in considering problems on columns; moreover, the formula is derived in a logical way, which theory shows to be a first and true approximation of the problem.

* *Bulletin No. 44*, University of Illinois, Table 7, p. 35.

a merit which, as far as the writer is aware, is not possessed by any other column formula. From the standpoint of the engineer, the use of theory, for the purposes of design, is to act as a guide, and it is essential to know within what limits it may be safely relied upon. In this respect, the derivation of the formula, as already pointed out, errs on the side of safety, and this is also true when applying it to evaluate the shear stresses set up in the column. Other formulas in use for the design of columns, such as the straight-line formula, can be modified to allow for secondary flexure. For instance, the straight-line formula can be put in the form,

$$p = f - a\frac{\rho}{t} - b\frac{l}{\rho}$$

where a and b are constants.

This formula, although useful within a limited range of $\frac{l}{\rho}$, for purposes of rapid design, can only be roughly modified to include eccentric and transverse loading; and, as a formula for co-ordinating the results of experimental tests, is of little value.

The writer does not claim that any great accuracy has been obtained in deducing the constants from the experimental data in the paper. The data given with the tests are not sufficient, and the range of the experiments is too limited to allow of more than first approximations being obtained. To dissect and co-ordinate the mass of experimental data now in existence is beyond the powers of any one man, and the work can only be carried through by a body such as the American Society of Civil Engineers, who, the writer understands, has formed a Special Committee to deal with the subject. A co-ordination of the experimental data on column tests would be of great value to the Engineering Profession, and the American Society of Civil Engineers is to be congratulated on being the first to take the matter in hand.

APPENDIX.

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AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

MAXIMUM STRESSES IN BASCULE TRUSSES.*

By W. Watters Pagon, Jun. Am. Soc. C. E.

To the best of the writer's knowledge, nothing has been published on the subject of the maximum dead stresses in the trusses of bascule bridges, of the trunnion, rolling lift, or combination types, and, as a result, several instances have come to his notice where the stress sheet showed only the dead stresses for the bridge closed and not the maximum stresses. In one case the true stress was three times the stress

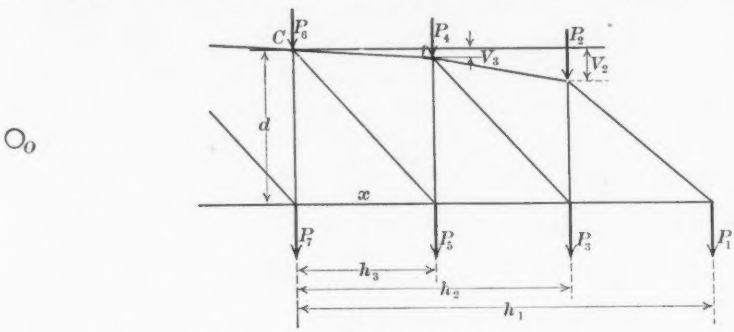

Fig. 1.

shown, though, fortunately, in this particular member, this was not the governing stress. There were several members, however, where the dead stress was the governing one, and in some of these the section had to be increased about 6 per cent.

Method of Determining Maximum Stresses.—Assume the truss shown in Fig. 1, subjected to the joint loads P_1, P_2, P_3, etc.; the

*This paper will not be presented at any meeting, but written communications on the subject are invited for publication with it in *Transactions*.

problem is to find the maximum dead stress in the lower chord member, x. Let the truss revolve about a horizontal axis at O. Find the dead stress in x with the truss in the horizontal position. Call this stress S_H. Find the dead stress in x with the truss vertical, or 90° from the first position. Call this stress S_V. Then the true maximum dead stress is $S = \sqrt{S_H^2 + S_V^2} = \frac{1}{d}\sqrt{M_H^2 + M_V^2}$, where M_H and M_V are the respective moments about the moment center, C. In parallel chord trusses without secondary system, $S_V = (P_1 + P_3 + P_5$, etc.$) \times d \div d$, equals the sum of the bottom chord loads between the member and the free end of the truss; but for parallel chord trusses with secondary system, S_V equals the sum of all the bottom chord and half of the middle joint loads between the member and the free end.

In the foregoing, attention has been devoted to bottom chord members only, for, although the theory applies to all members, the top chords and web members* receive their maximum dead stresses when the truss is in the horizontal position. In any special case, it will be easy to determine which members need to be investigated for maximum stress by the use of the following criterion, which is easy to remember and simple to apply.

Criterion for Maximum Stress.—Whenever S_H and S_V for any member are of like sign, there will be a maximum dead stress, greater than either, to which the member will be subjected during every opening and closing. When this stress occurs, the bridge will be open at an angle $\tan.^{-1} \frac{S_V}{S_H}$, with the horizontal.

Whenever S_H and S_V are of unlike sign, the maximum stress will be one of the two, whichever is the larger (usually S_H).

Summary.—To find the maximum dead stress in any member in a truss revolving about an axis perpendicular to it: Find S_H for the bridge closed. Find by inspection whether S_V will have the same sign as S_H. If of like sign, find S_V and then the maximum stress from $S = \sqrt{S_H^2 + S_V^2}$. If of unlike sign, S_H (or S_V) is the maximum stress.

As the methods and theory given are perfectly general, any two directions at right angles to each other may be substituted for

* For web members this is strictly true only in parallel chord trusses.

"horizontal" and "vertical." They may also be applied to any revolving truss subjected to a number of parallel loads which are not vertical.

Theory.—In Fig. 2, which shows the truss open at an angle, α, let the loads and dimensions be the same as in Fig. 1. To find the stress in x for this position:

$$S' = \frac{M}{d} = \frac{(P_1 \cos.\ \alpha\, h_1 + P_3 \cos.\ \alpha\, h_2 + \ldots\ldots P_2 \cos.\ \alpha\, h_2 + \ldots\ldots \text{etc.})}{d}$$

$$+ \frac{(P_1 \sin.\ \alpha\, d + P_3 \sin.\ \alpha\, d + \ldots\ldots \text{etc.} + P_2 \sin.\ \alpha\, v_2 + \ldots\ldots \text{etc.})}{d}$$

$$= \frac{\Sigma P\, h \cos.\ \alpha + \Sigma P\, v \sin.\ \alpha}{d} = \frac{M_H \cos.\ \alpha + M_V \sin.\ \alpha}{d}$$

$$= S_H \cos.\ \alpha + S_V \sin.\ \alpha.$$

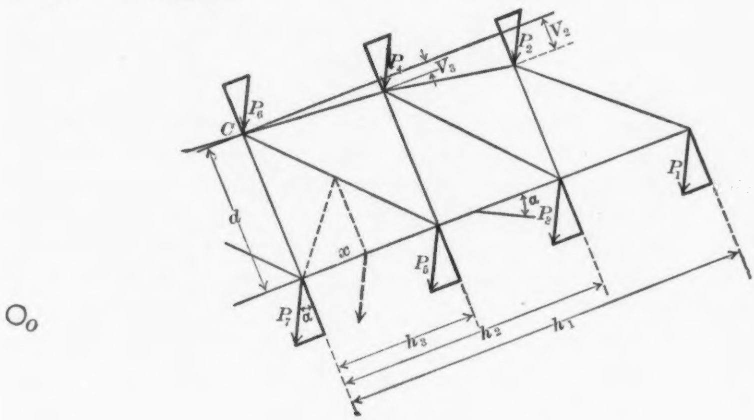

FIG. 2.

In order to find the maximum value of S', differentiate as to α:

$$\frac{dS'}{d\alpha} = -\, S_H \sin.\ \alpha_o + S_V \cos.\ \alpha_o = 0, \text{ therefore } \tan.\ \alpha_o = \frac{S_V}{S_H};$$

and $\sin.\ \alpha_o = \dfrac{S_V}{\sqrt{S_H^2 + S_V^2}}$, $\cos.\ \alpha_o = \dfrac{S_H}{\sqrt{S_H^2 + S_V^2}}$;

therefore $S = \dfrac{S_H^2}{\sqrt{S_H^2 + S_V^2}} + \dfrac{S_V^2}{\sqrt{S_H^2 + S_V^2}} = \sqrt{S_H^2 + S_V^2}$

= the maximum, S'.

For any member in a secondary system, the same reasoning holds true.

To find a criterion for the position giving the maximum stress, lay off, in Fig. 3, $ab = S_H$, horizontally and measured to the right, if in compression. Then lay off $bc = S_V$, upward if in compression, making abc a right angle. Then $ac = \sqrt{S_H^2 + S_V^2} = S$. Draw a line, ad, at an angle, α, with ab. Project b to e, and c to d. Then $ae = S_H$ cos. α, and $ed = S_V$ sin. α; and therefore $ad = S'$ for the angle, α.

Draw a circle through a, b and c. Then, as $abc = 90°$, ac is a diameter, and as adc is a right angle, d lies on the circle. Therefore the length of any chord, ad, at an angle, α, with the horizontal, is the stress, S'. The greatest chord is the diameter, ac, which makes an angle, $\tan.^{-1} \dfrac{S_V}{S_H}$.

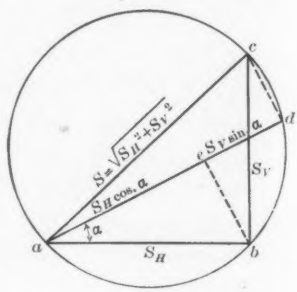

FIG. 3.

This diagram, Fig. 3, completely represents the stress in any bottom chord member. For other members similar circles may be drawn. It is easy to see that for every member having stresses, S_H and S_V, of the same sign, the circle will be drawn as shown, and the maximum stress will be attained at each opening. For other members, the circle will lie in another quadrant, as a is the origin; and therefore this stress will never be attained. Therefore S_H (or S_V if it is the greater) will be the maximum for which provision is to be made.

AMERICAN SOCIETY OF CIVIL ENGINEERS

PAPERS AND DISCUSSIONS

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CONSTRUCTION OF A HIGH-SERVICE RESERVOIR AT BALTIMORE, MD.

By P. A. Beatty, M. Am. Soc. C. E.

To be Presented October 16th, 1912.

The new high-service reservoir, Lake Ashburton, constructed by the Water Department of Baltimore in 1908-10, is on a site near the intersection of Slingluff Avenue and Liberty Road, and only a short distance west of the Western Maryland Railroad. The spot selected is between two low spurs which run eastward from a plateau, about 450 ft. above mean tide, and terminate in the low ground at Pecks Run. Between these spurs there flowed a small stream, Williams Run, carrying the cesspool overflow from a small suburban development and the storm-water flow from a few hundred acres.

Sub-surface investigations indicated that, underlying a foot of top soil, there was a stratum of clay, from 1 to 4 ft. in thickness, beneath which the formation consisted of "rotten rock" (a decomposed serpentine) of varying depth, blanketing and lying between ledges of semi-decomposed rock and hard serpentine, more or less shattered, and with occasional nests of boulders. A portion of the site which was in timber was cleared and grubbed under a separate contract.

The work to be done consisted of the construction of an earthen dam, and the excavation of a basin, with appurtenant structures; and also the construction of a concrete storm-water sewer for the diversion of Williams Run.

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

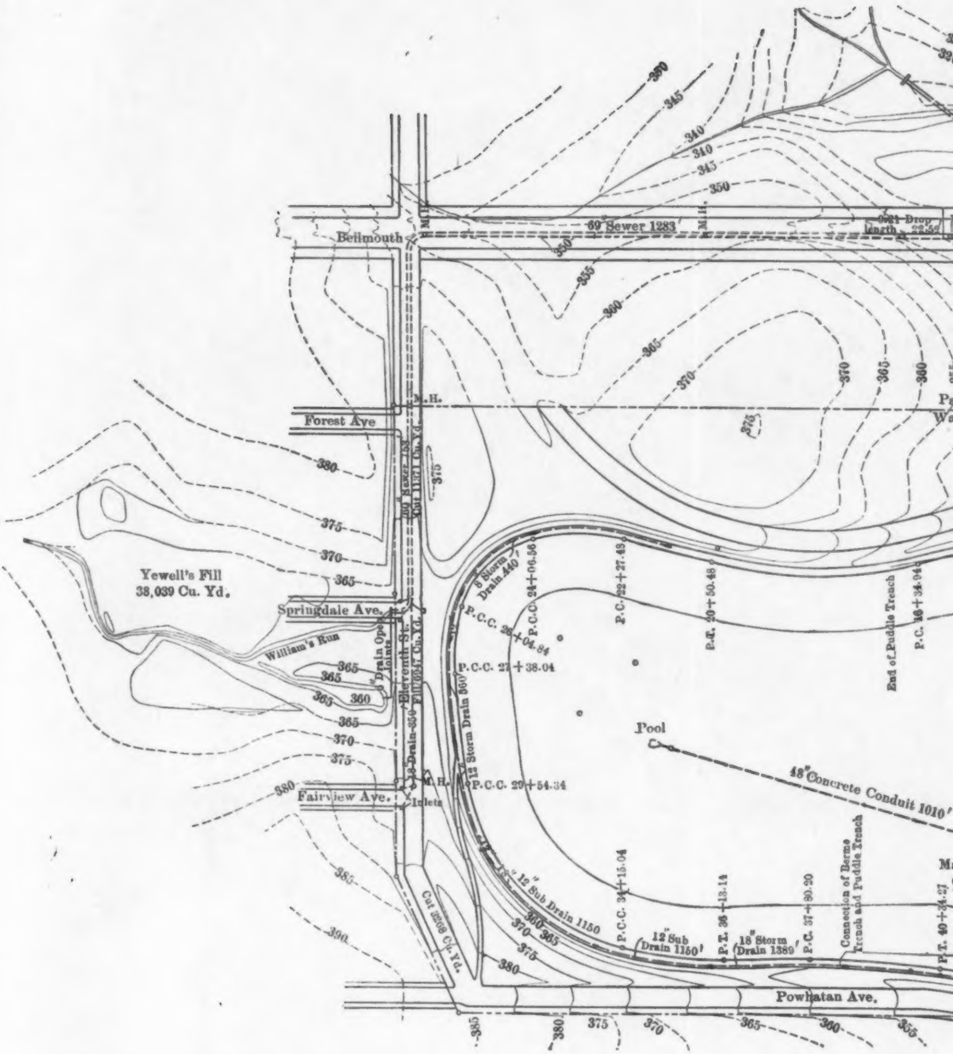
The work was let on a unit price basis, the reservoir work to Lane Brothers Company and Jones, of Altavista, Va., and Baltimore, the time being limited to 400 working days; and the diversion sewer to C. B. Clark and Company, a local firm, the limit being 200 working days. In the ten bids received for the construction of the reservoir, the total prices ranged from $494 810 to $837 241, and the working time from 200 to 700 days. In the thirteen bids received for the construction of the diversion sewer, the total prices ranged from $33 572 to $73 986, and the working time from 100 to 650 days.

Except for some topographical modifications, the work was completed substantially as laid out originally. Between the Western Maryland Railroad and Ninth Street the grade of Liberty Road was raised, and Slingluff Avenue was brought to the grade of Liberty Road on a curved fill with a 4% grade. This necessitated the construction of a 6 by 6-ft. box culvert, the extension of the diversion sewer 98.3 ft., caring for the flow from an existing box culvert, and the construction of a 9½-ft. arch culvert 240 ft. long to carry Pecks Run. The latter was built by the Baltimore Sewerage Commission. These modifications are shown on Plate LXVIII, which covers the work as completed.

The original plans provided for a dam to be built in 6-in. lifts, thoroughly rolled, and for a reinforced concrete diaphragm against the outer wall of the cut-off trench, the trench to be back-filled with rolled or rammed clay. The former was changed to a fill deposited under water; the diaphragm in the cut-off trench was eliminated, and the trench was back-filled with clay, deposited under water.

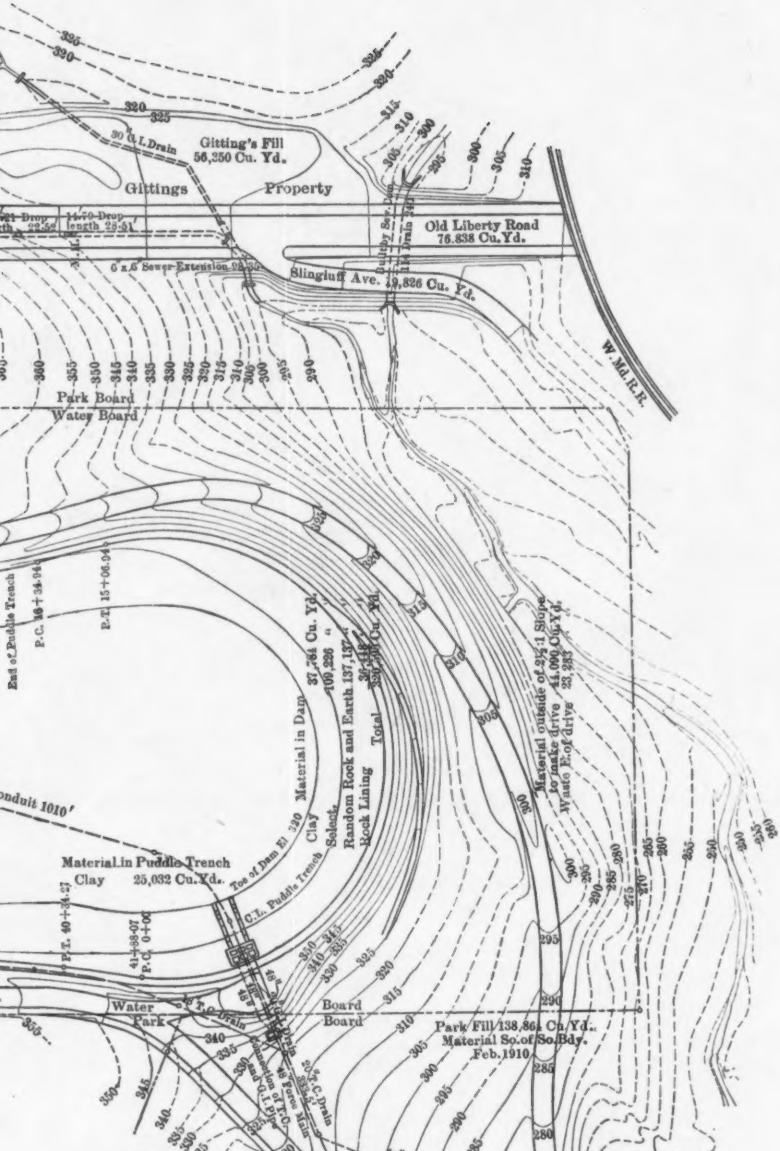
Ground was broken on July 6th, 1908. The soil within the basin area was removed with wheeled scrapers and stored along the margins of the Water Board's property where convenient, after which the clay was similarly removed and stored near the soil banks, or within the basin area, adjacent to the cut-off trench. Meanwhile, that portion of the dam site which lay in the valley was being cleared of boulders and stripped of loam, roots, muck, etc., this waste material being deposited beyond the toe of the dam proper, and in the base of the East Drive fill. Several small springs, discovered near the bottom of the valley in the dam foundation, were led beyond the toe in blind drains.

Williams Run was diverted at the point where it crossed Contour 320, and was carried along the side of the south spur, crossing the



MAP OF
ASHBURTON RESERVOIR

PLATE LXVIII.
PAPERS, AM. SOC. C. E.
AUGUST, 1912.
BEATTY ON
HIGH-SERVICE RESERVOIR.



dam site at Elevation 315, being conducted over the cut-off trench in a 4 by 4-ft. box.

The cut-off trench was excavated with plows and scrapers, and with carts on the hillsides where it could be benched, derricks being set up to cover the work as the excavation deepened. The material through which the trench was sunk consisted of rotten rock, semi-decomposed rock, hard seamy rock, and hard serpentine ledges. At one point there was a mass of boulders, many of them containing 1 cu. yd. or more, extending from the surface to bed-rock and bedded in a matrix of gray, fully decomposed rock. On the north side, the trench cut a dike of felsite, which crossed it nearly at right angles, was about 18 ft. thick between nearly vertical faces, and extended from the surface to the bottom of the trench, about 62 ft. deep at this point.

When bed-rock was reached, it was found to be more or less seamy in places, making it necessary to continue the excavation until the conditions became satisfactory. Very little water was encountered at any point, the few small springs exposed being led to 2-in. stand-pipes through smaller pipe connections, and these stand-pipes were ultimately grouted. The average depth of the trench was 40 ft., the length 2 100 ft. The timbering consisted of 2 by 10-in. sheeting, 6 by 8-in. rangers 4 ft. 6 in. from center to center, and 6 by 8-in. braces 8 ft. apart.

In August a shovel was placed in Eleventh Street, and the fill at Williams Run was made. The water was allowed to pool to a depth of about 10 ft. on the upper side, and was carried thence through two lines of 18-in. pipe through the south end of the fill. Later, the pool was filled from the Eleventh Street cutting. When the run was finally diverted through the sewer, the pipes were followed into the fill and removed as far as practicable; they were then plugged with concrete, and a mass of clay was tamped into the excavation. An excavation was made afterward in the fill to the west of Eleventh Street, and a sub-drain was laid in a shallow trench in the original soil, well surrounded with broken rock, the purpose being to prevent the collection of a pool of stagnant water in the fill and the attendant danger of pollution of the water in the reservoir.

Eleventh Street being completed, the shovel was removed to the basin and began work on the north side, following down the slope with a series of curved cuts until Elevation 355 was reached, where a 15-ft.

berm was left; thence, the shovel followed the slope and shape of the reservoir to the bottom, at Elevation 320, after which successive parallel cuts were made until the cutting of Shovel No. 2 was joined.

Before the dam site was prepared for rolling, this shovel stored select material on the hillside immediately east of the cut-off trench, the random rock and earth being deposited in the base of the driveway fill, and tracks being laid across the valley on an embankment which had been built with carts.

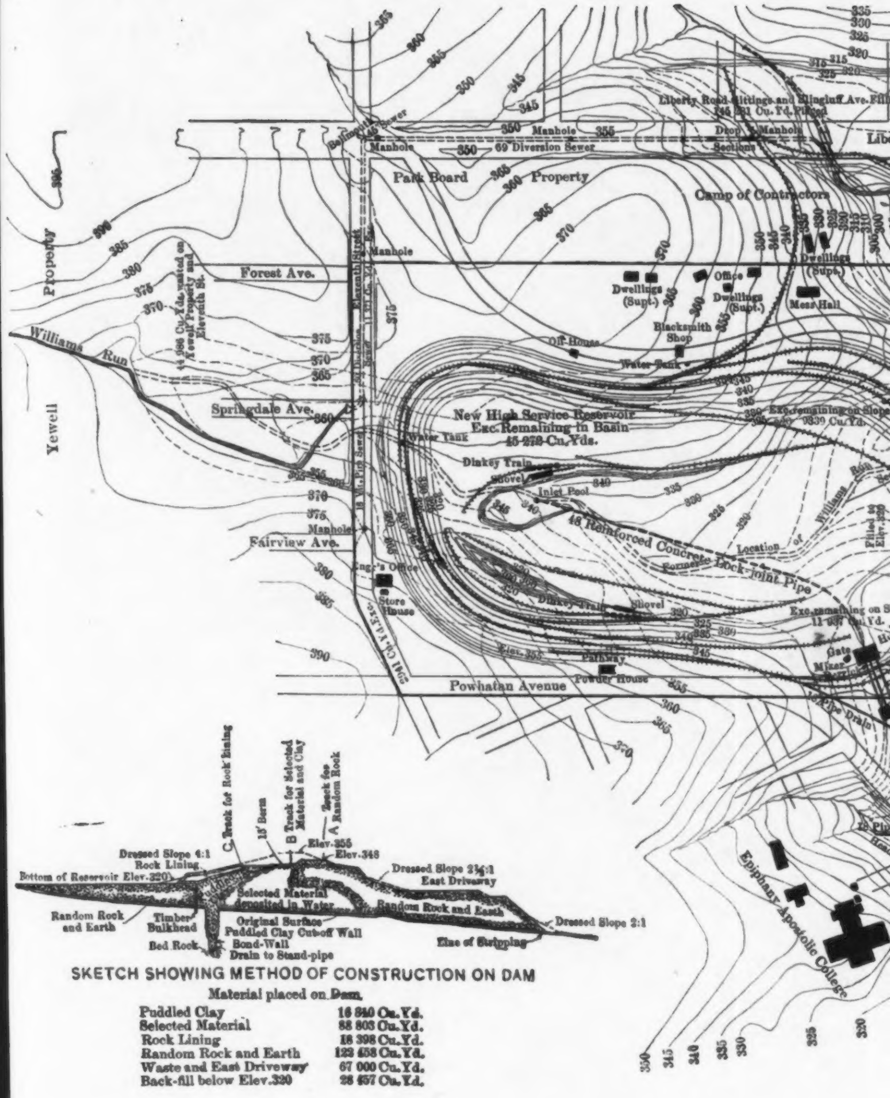
On the north side, the first layout of tracks from the shovel ran eastward on a descending grade on the hillside, crossing the cut-off trench on a bridge, and curved northward through a shallow cut in the end of the spur to a switchback near the intersection of the north boundary line with Contour 315, from which the cart fill just mentioned was reached by a sharply falling grade. As this fill increased in height, the grade of the switchback became flatter and ultimately was reversed, giving a steep grade from the switchback to the dam. It was then abandoned, and the tracks were laid as shown on Plate LXIX.

As the shovel descended the slope, switchbacks were necessary to get the spoil out of the basin. Plate LXIX shows the loading track running to a switchback in the eastern end of the basin, from which the running track ascends the slope westward to a second switchback at the west end, at Elevation 350; thence it ran eastward, crossing the berm and curving northward on a descending grade to Liberty Road. Near the water tank, Track "C" curved around the inner slope of the dam. A second track, "A," gave access to the outer slope.

Plate LXIX shows the Liberty Road and Gittings fills completed, and the Slingluff Avenue fill nearly finished, with the dam brought to Elevation 345.

Shovel No. 2 was started at Contour 320 on the south side, having first made a cut through the south spur immediately east of the gate-house site, through which a running track was laid. In the first position, with the shovel and loading track on the reservoir floor, the track reached the cut east of the gate-house by an ascending grade, from which point, with a descending grade, it curved around the end of the spur and reached the dam through the East Drive cut.

A switchback, near the south boundary line, gave access to the fill on the Park Board's property south of the gate-house. With this

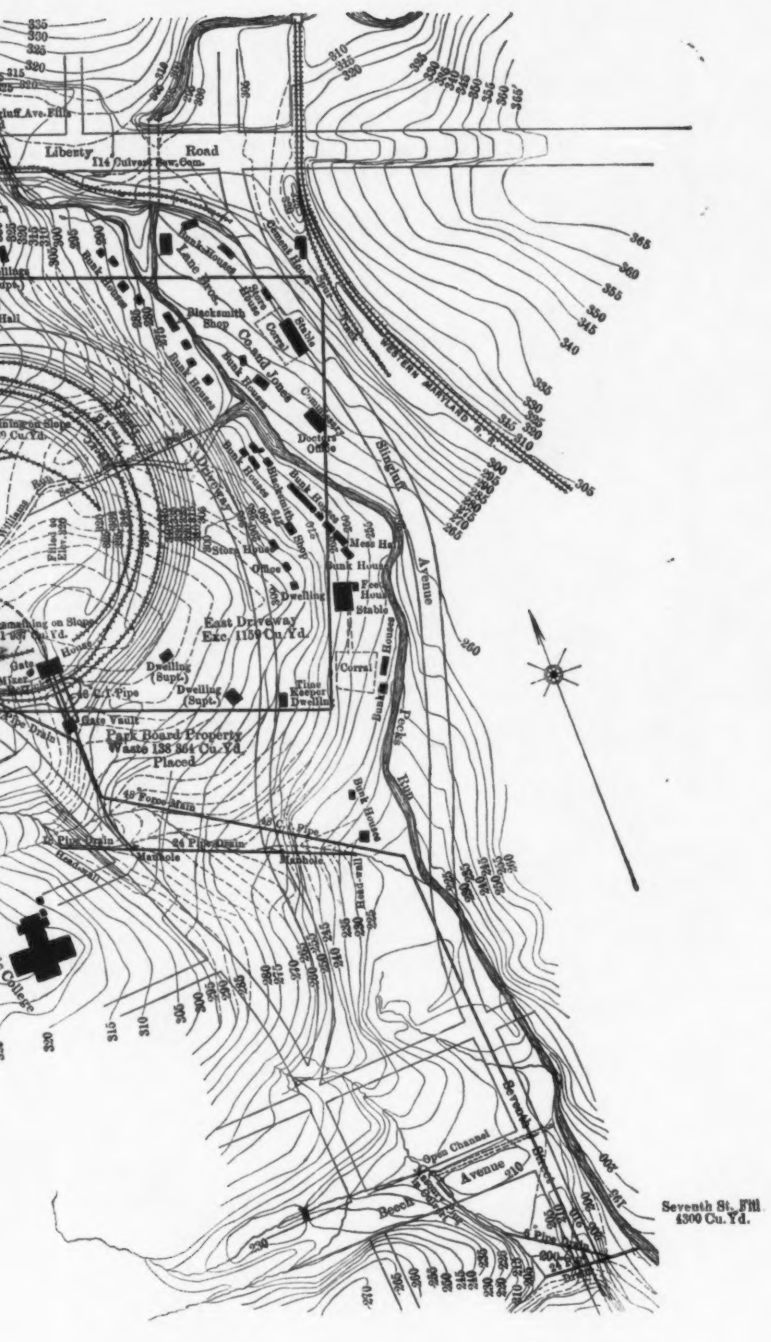


SKETCH SHOWING METHOD OF CONSTRUCTION ON DAM

Material placed on Dam.

Puddled Clay	16 840 Cu. Yd.
Selected Material	88 803 Cu. Yd.
Rock Lining	18 398 Cu. Yd.
Random Rock and Earth	123 458 Cu. Yd.
Waste and East Driveway	67 000 Cu. Yd.
Back-fill below Elev. 320	28 457 Cu. Yd.

SKETCH SHOWING
METHOD OF CONSTRUCTION OF
NEW HIGH-SERVICE RESERVOIR
FOREST PARK BALTIMORE MD.


Liberty
Road
Bunk Houses
Lane Bros. Co. and Jones
Blacksmith Shop
Corral
Stable
Cement House
Western Maryland R. R.
Commissary
Doctors Office
Slingluff
Avenue
Mess Hall
Store House
Office
Dwelling
Stable
East Driveway
Exc. 1150 Cu. Yd.
Dwelling (Supt.)
Time Keeper Dwelling
Corral
Gate Vault
Park Board Property
Waste 138 864 Cu. Yd.
Placed
48 Force Main
48 C.I. Pipe
24 Pipe Drain
Manhole
Head-wall
Pecks
Run
Seventh Street
Open Channel
Avenue
Beech
Seventh St. Fill
4300 Cu. Yd.

location of the tracks, the dam was brought to Elevation 300, and the first lift of the fill on the Park Board land was made. Two relocations of tracks were necessary, the last being that shown on Plate LXIX, from which position the dam and the last lift of the Park fill were completed, and the fill for Powhatan Drive was made. The last relocation was made when Shovel No. 2 was moved from the floor of the basin to the top and began to work down the slope, which was rendered necessary, in part, by the scarcity of proper material for construction in the lower cut.

Plate LXIX shows that Shovel No. 2, having worked down the slope, is again on the reservoir floor, removing the long, narrow ridge between the foot of the slope and the previous cutting. The track runs from the shovel to a switchback near the west end of the basin at Elevation 340, thence eastward along the slope to a point near the gate-house, where it bifurcates, one branch being continuous with Track "A" from the north side, the other, "B," lying between "A" and "C." "A" is in position for raising an outer embankment of random rock and earth; "C" can place either "rock lining" or clay; while "B" is just completing a lift of select material. The fill on the Park land is shown as completed, the terraces indicated by the contours being dressed to a regular slope later.

The necessity of getting out certain classes of material as they were needed required considerable moving of the shovels back and forth, but less, on the whole, than might have been expected, as it was usually possible, by careful planning, to have the shovels working in the materials needed, and thus very little time was lost.

The total quantity removed from the basin below the clay was 582 458 cu. yd., of which 161 307 cu. yd. were earth, 98 752 cu. yd. were loose rock, and 322 399 cu. yd. were solid rock. The only unusual features connected with the construction were the methods used on the dam and the cut-off wall. A short description of the excavation of the trench for the cut-off wall has been given. When the question of the elimination of the reinforced concrete diaphragm was taken up, a series of experiments was undertaken with rammed clay and clay deposited in water. Clay and rotten rock, rammed in cylinders and placed under water pressure, were tested for penetration. When compressed in cylinders, under a pressure of 150 lb. per sq. in., the clay weighed 131.6 lb. and the rotten rock 129 lb. per cu. ft.

Several 9 by 12-in. boxes, 8 ft. long, constructed so that the sides could be removed by loosening nuts and rods, were placed on end and filled with water. Clay was shoveled into some of these, and rotten rock into others, and some were filled with mixtures of the two in varying proportions. In several of the boxes a number of clods and lumps of clay and rotten rock were placed. About two weeks later, the sides of the boxes were removed and the contents examined, and found to be compact and homogeneous, the clods having fused with the surrounding material and practically disappeared. The material was compact and tough, and only slightly damp. The clay weighed 125 lb. and the rotten rock 118 lb. per cu. ft. This material had been subjected to no extraneous pressure; in the dam, the weight would certainly be very much greater, owing to the pressure exerted by the superincumbent material.

A section of the trench having been completed, several feet of clay were rammed into it in 3-in. layers, a picked foreman having charge of this work. It presented a good appearance, and seemed to be quite solid. Several 20-ft. lengths of 2-in. pipe were then set vertically at various points, their lower ends being deeply sunk in the clay (which was firmly rammed around them). Then these pipes were filled with water. The water usually forced its way to the surface around the pipes, but in several cases found lines of weakness in the clay and appeared several feet away. Several pits were also sunk in the clay and filled with water; two or three of these, placed directly beneath the braces, leaked, showing that the ramming was imperfect at these points. This led to the conclusion that satisfactory results could not be obtained by ramming. The cost of this work was 87.4 cents per cu. yd. It was then decided to fill the trench with clay deposited in water, using a certain percentage of rotten rock where practicable, care being taken to insure thorough mixing.

It was impracticable to remove the sheeting, but every fifth or sixth plank was cut away as the clay arose, permitting the puddle to fill the spaces between the planking and the trench walls. Heavy vertical timbers were placed at short intervals and braced hard against the rangers. The lower rangers were held by the cantilever ends of the verticals, and the original bracing was removed as the clay reached it. Usually, two sets of cross-braces were removed before the verticals were shifted. When the clay neared the third set, the jack-braces

FIG. 1.—CUT-OFF TRENCH AT INNER TOE OF DAM, LOOKING NORTH.

FIG. 2.—LOOKING NORTH OVER RANDOM ROCK SECTION OF DAM FOUNDATION. TO LEFT, EMBANKMENT AT TOE OF SELECT SECTION. TO RIGHT, EMBANKMENT ON EAST DRIVEWAY FILL. DEPRESSION TO BE FLOODED AND FILLED WITH RANDOM ROCK AND EARTH.

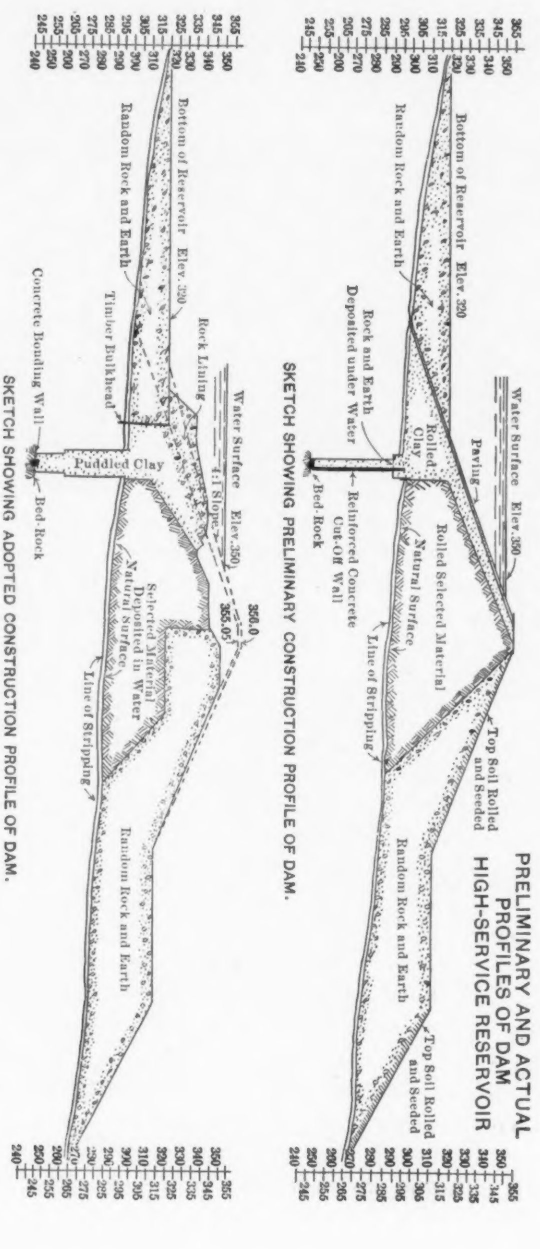

PRELIMINARY AND ACTUAL PROFILES OF DAM HIGH-SERVICE RESERVOIR
Bottom of Reservoir Elev. 320
Random Rock and Earth
Water Surface Elev. 350
Paving
Rolled Clay
Rock and Earth Deposited under Water
Reinforced Concrete Cut-Off Wall
Bed-Rock
Rolled Selected Material
Natural Surface
Line of Stripping
Top Soil Rolled and Seeded
Random Rock and Earth
Top Soil Rolled and Seeded
SKETCH SHOWING PRELIMINARY CONSTRUCTION PROFILE OF DAM.
Bottom of Reservoir Elev. 320
Random Rock and Earth
Rock Lining
Timber Bulkhead
Concrete Bonding Wall
Water Surface Elev. 350
4:1 Slope
Puddled Clay
Bed-Rock
Selected Material Deposited in Water
Natural Surface
358.0
355.05
Line of Stripping
Random Rock and Earth
SKETCH SHOWING ADOPTED CONSTRUCTION PROFILE OF DAM.

FIG. 1.

were eased off and the verticals were lifted and rebraced, auxiliary braces being placed where required during this change.

Through the deeper part of the trench a concrete toe-wall had been placed. This contained the drainage pipes from the springs, leading to the stand-pipes. This wall was not carried the entire length of the trench, the irregularity of the bottom seeming to form a sufficient obstruction to the passage of the water.

In placing the clay, picked foremen were employed, and they were constantly supervised by the inspectors. Care was taken to remove all foreign matter, and lumps were broken to the size of an orange, all stones of greater size being rejected. The shovelers were directed to cast the clay and rotten rock so that it would scatter in falling.

When rotten rock was accessible, from 40 to 50% was used. It was cast in from one side while the clay was being cast in from the other. The desired proportions were obtained by regulating the number of shovelers on each class of material.

The water was usually about 4 ft. deep at the beginning of each lift, and the casting was stopped when the clay was within about 1 ft. of the surface of the water. Fine roots, chips, cinders, etc., not noticeable in the clay, floated to the surface, and were kept skimmed off.

As the puddle reached the ground surface in the lowest part of the valley, clay bulkheads, several feet in height, were built across the trench, water was turned in, and a new lift was begun. In this way the puddle was carried up the hillsides in successive terraces, filling the trench to the surface throughout. Very little trouble was experienced from the slipping of these dry clay bulkheads, although founded on freshly puddled clay; there was usually some movement, accompanied by a slight heaving of the clay below the bulkhead, but it was so small that it was no consequence. The city paid 25 cents per cu. yd. for the clay thus deposited, which covered all charges from the storage pile to its place in the trench, and the placing and removal of the timbering rendered necessary by this method. The clay settled about 2 ft. in two or three months, but little settlement took place thereafter. Investigations made several months later showed that the clay was compact, firm, and slightly moist at a depth of 7 ft. below the surface.

The first fill across the valley was at the toe of the East Drive,

FIG. 1.—LOOKING EASTWARD FROM NORTHWEST CORNER. TERRACED CUTTING BY NORTH SHOVEL.

FIG. 2.—LINING WITH CLAY. TRACK ON COMPLETED LIFT OF SELECT SECTION. BACKGROUND, NEXT LIFT OF RANDOM ROCK. TO LEFT, RESERVOIR FLOOR, ELEVATION 320.

FIG. 1.—DRESSING INNER SLOPE OF DAM. THE ADDITIONAL MATERIAL REINFORCING THE ROCK LINING AGAINST SLIPPING CAN BE SEEN DISTINCTLY.

FIG. 2.—LOOKING WESTWARD FROM DAM. LOCK-JOINT INFLUENT PIPE IN CENTER.

and was made with carts; on this a track was laid, connecting with the switchback on the north side, and later joining the track from Shovel No. 2, passing through the cut on the East Drive south of the dam.

A second embankment was built across the valley at the line of junction of the select and random rock sections, and Williams Run was turned into the depression thus caused, forming a pool, into which the random rock and earth were dumped.

There was not enough earth in the filling material to make an impervious embankment, nor was this desired, the intention being to obtain a compact but somewhat pervious fill, which would allow the escape of any water percolating through the select section of the dam. This method was followed on the random rock section until the narrowing of the dam, as the height increased, rendered it impracticable.

The question of building the entire dam by depositing the materials in water had been under discussion for some time, and it was finally decided to adopt this method, the contractors making a satisfactory rebate on the cost of this portion of the work. It was expected that the principal difficulties would be the placing of the materials under water in their designated positions and the preservation of the proper slope on the inner side.

The following method was adopted: The inner slope of the random rock section was given a lining of select material dumped from cars, to prevent the escape of water from the proposed pool; and a narrow embankment was built across the valley just outside of the cut-off trench, to prevent the water from entering the latter.

This basin was then flooded, and select material was dumped along the outer edge of the pool, the track being thrown inward as the fill widened. The tracks had to be kept 2 ft. or more above the surface of the water, the fill showing a tendency to slip out, particularly where the water had considerable depth, and the submerged portion took a very flat slope, the finer portions of the select material running far ahead and forming a bed of mud, in which the solid materials were thoroughly bedded.

This tendency of the toe to advance had to be watched carefully, in order to prevent it from encroaching on the clay section, a difficulty which was overcome later by raising a low ridge of clay on the line between the select and clay sections. A surprising quantity of air

seemed to be entrained in the select material, and somewhat less in the clay. That this was expelled at an early date seems conclusive, in view of the small amount of settlement in the finished structure.

The clay having reached the top of the cut-off trench, a wooden bulkhead was erected, 12 ft. from its inner edge, on the line of the proposed extension of the clay above the ground surface and the fill inside of the dam, which was being placed to bring the floor of the reservoir to Elevation 320.

A trench was excavated to a depth of several feet, in which a continuous sill piece was laid, to which posts, 6 ft. from center to center, were fastened with ½-in. iron pins passing horizontally through both posts and stringers. The posts were braced in a vertical position, and the trench was packed with rock. To these posts 2-in. planks were spiked as the fill was raised.

The first lift of the select section having reached the clay line, clay was brought from the storage banks, being loaded on the trains with scrapers by means of a trap. This clay was dumped on the edge of the select embankment and cast into the water, the same care being taken as in filling the cut-off trench, the limit of the clay puddle being the bulkhead.

Meanwhile, the random rock and earth section had been started on the next lift and lined as before to form the outer embankment of the next pool, the depression was again flooded, and the foregoing operations were repeated. There was considerable difficulty in keeping the bulkhead standing, but it was accomplished by judicious dumping on the inside to counterbalance the pressure of the clay. These operations were repeated on each lift.

Above Elevation 320 it became necessary to raise an embankment on the inner as well as on the outer side, on a 3:1 slope, to conform to the profile. For this purpose, the best rock that the shovels could reach was used to form the "rock lining," the first lift being deposited at the intersection of the 3:1 slope with the floor at Elevation 320, and reaching from hill to hill. This embankment was 5 ft. high, and just sufficiently broad on top to hold a track. A light blanket of clay was dumped on the pool side and a new pool was formed. Water was delivered to the dam site by a 6-in. main connected with the city supply. From this elevation up the pools were usually 3 or 4 ft. deep, the lifts being about 5 ft. Each successive lift of rock lining over-

FIG. 1.—SWALE AND BERM AT ELEVATION 355. DAM IN BACKGROUND.

FIG. 2.—LOOKING SOUTHWARD ACROSS BASIN. DAM AND GATE-HOUSE IN BACKGROUND.

lapped the clay sufficiently to give the required slope, and this operation was repeated until the top of the dam was reached.

In order to insure a tight job at the gate-house, the space between the walls of the foundation pit and the concrete was filled with clay puddle, and a mass of clay, from 8 to 10 ft. in thickness, was placed entirely around the structure, extending from the foundation to above the water line. This was brought up with the successive lifts of the dam.

Everything progressed in a satisfactory manner until an elevation of about 335 was reached, when, a little north of the middle of the dam, a movement was observed in the rock lining. At the time, rock lining was being placed, and a considerable quantity had been dumped above the place in question. The movement consisted of a steepening of the slope a short distance above the toe, covering a space of 200 ft. or more, and a swelling of the surface. As soon as this came to the writer's notice, the dumping was stopped at this point, and the track was thrown down to the part affected. Rock was first dumped over the toe to buttress it and prevent kicking out, after which the track was thrown up the slope, and a good weight placed on the swelling. By midnight, it was thought that the danger was over, and the men quit work. In the morning, as early as possible, the Water Engineer and his assistant made a thorough examination, and found that for about 500 ft. the toe of the rock lining, Elevation 320, had moved inward from 1 to 3 in., the slope appearing to be unaffected except at the point mentioned.

It was decided to buttress the toe from side to side of the valley, and weight the slope with a gradually diminishing mass of rock, giving it a 4:1 slope between the hills, warping back to a 3:1 slope as the ends were approached. A line of centered stakes was placed across the disturbed portion of the dam and kept under observation for several months, but no additional movement was observed, nor has there been any sign of movement in any portion of the dam since.

In June, 1909, several test-pits were sunk in the select material between the rock section and the pool for the purpose of examining the condition of the material which had been deposited under water, and to get an idea of the quantity of seepage. These pits penetrated two lifts, reaching a third, the bottoms being 7 or 8 ft. below the surface of the pool. No. 1 pit, 12 ft. from the edge of the pool, on

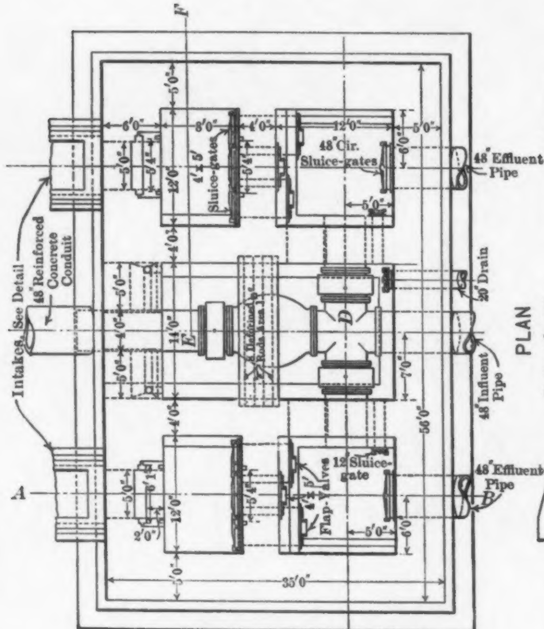
the third day contained 2 ft. of seepage. Nos. 2 and 3, 35 ft. from the pool, remained dry. No. 4, 10 ft. from the pool, on the third day contained 3 ft. of water, and the bottom showed a tendency to heave. Later, the pool was drained, and an examination was made of the bottom, which consisted of gray mud in which matrix all the stones were thoroughly bedded.

The dam was finished at Elevation 356, 1 ft. above the grade called for by the plans, the last 3 ft. being placed dry. The extra foot was added for settlement. It seems, however, that the settlement was about completed before the crest was placed. Stakes were set every 50 ft., and observed for a year, during which period the maximum observed settlement was 0.2 ft. It was impracticable to ascertain the shrinkage during construction, as the entire surface was being worked over, affording little opportunity to maintain a permanent bench.

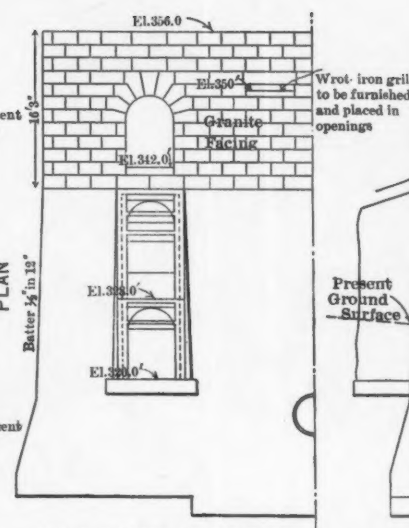
The total quantities placed in the dam were: 37 784 cu. yd. of clay; 109 226 cu. yd. of select material, and 137 137 cu. yd. of random rock and earth. The East Driveway fill, outside of the dam, contained 67 373 cu. yd. of random material, and the cut-off wall, 25 032 cu. yd. of clay.

At the west end of the reservoir, a cut-off trench was excavated well into the rock, with an average depth of 20 ft., and extending into the hills to the north and south. This was filled with puddled clay, which was brought up as a core-wall to the berm level, random material being placed on either side to support it above the natural ground. The Liberty Road fill was begun from a trestle, erected along the south side of the road, the caps and stringers being removed later. In the track giving access to this fill, 132 800 cu. yd. were placed. A bench was cut along the south slope of the fill, corresponding to the grade of Slingluff Avenue, and 18 000 cu. yd. were used in connecting the two streets. West of Eleventh Street 38 000 cu. yd. were wasted in the Williams Run valley, 159 000 cu. yd. were placed on the Park property south of the gate-house, and 2 800 cu. yd. were placed in the basin to bring the floor to Elevation 320.

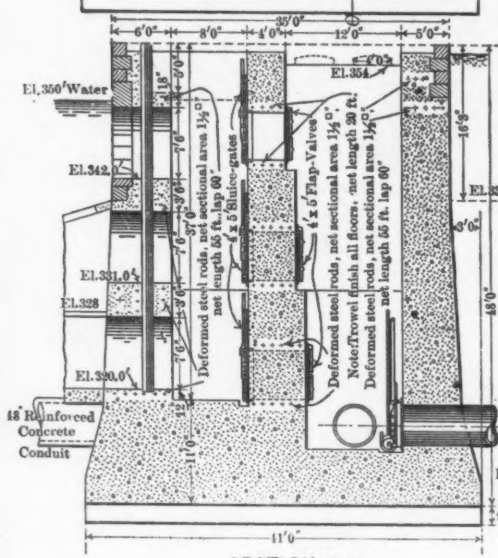
To take care of the surface and sub-surface flow which would approach the reservoir from the west end, a trench has been excavated at the outer edge of the berm, and extends from a point near the north end of the dam around to meet the cut-off trench at the south end, with the bottom on a 5% slope. Along the outer side of this



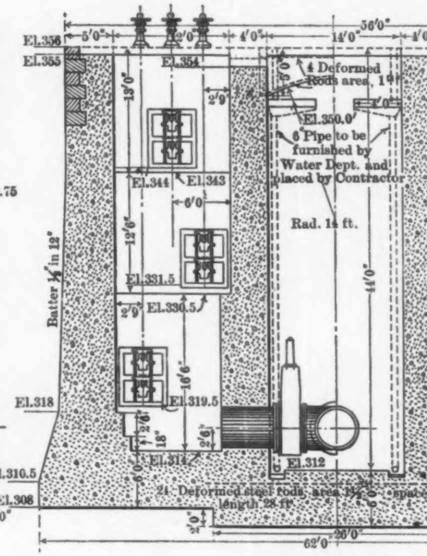
PLAN



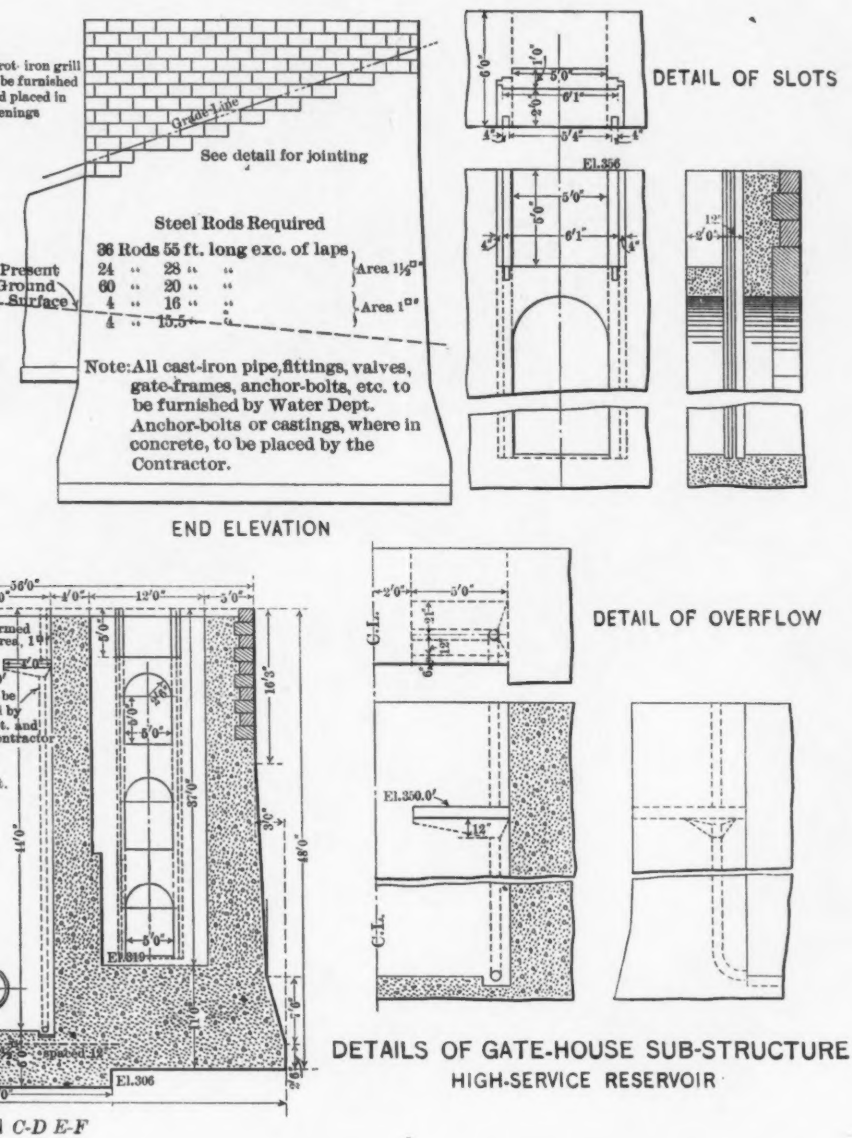
HALF FRONT ELEVATION



SECTION *A-B*



SECTION *C-D E-*



DETAILS OF GATE-HOUSE SUB-STRUCTURE
HIGH-SERVICE RESERVOIR

trench there is a 6-in., open-joint sub-drain, surrounded with broken stone, to intercept any seepage which might reach the trench; 3 ft. above this there is laid a line of pipe, connecting with the surface by intakes placed 250 ft. apart, to carry off all surface water flowing toward the reservoir. The trench was back-filled with clay. The two lines of pipe discharge into a manhole west of the gate-house, from which there is a line of pipe connecting with the 20-in. drainage line from the gate-house, running to a manhole on the Park property, whence the flow passes through a 24-in. pipe to a point near the bank of Pecks Run.

Beginning at the north face of the gate-house, the first joint being concreted into the wall, and making connection with the 48-in. force main, a 48-in. reinforced concrete, lock-joint pipe (Meriwether system) extends 1 010 ft. to a pool near the western end of the basin, forming a continuation of the force main. This conduit is laid in the reservoir floor, and crosses the cut-off trench on a reinforced concrete beam. A concrete collar has been placed around the pipe at the cut-off trench.

The force main passes through the central chamber of the gate-house, where there is a check-valve and a four-way branch, and is controlled by a gate-valve. Between the gate-house and the gate-vault, there are three lines of 48-in. pipe. These are connected by a four-way branch in the gate-vault, where there are additional gate-valves. The west line extends through the south wall of the gate-vault, and is closed with a cap. The central line—the force-main—extends to the North Avenue main, which is laid in Seventh Street after leaving the Park property.

From the central chamber of the gate-house, a line of 20-in. cast-iron pipe, for drainage purposes, extends to a point south of the gate-vault, which it also drains through a 2-in. pipe connection; from this point the line is of double-strength vitrified pipe, laid in concrete. Through this pipe and a blow-off at the lowest point on the force-main the reservoir is drained.

The gate-house substructure, extending from Elevation 308 at the foundation to Elevation 356 at the floor level, contains a central and four side chambers. The four-way branch in the central chamber, connects with the south side chambers, and is controlled by gate-valves, operated from the central chamber. The side chambers discharge through the two lines of 48-in. effluent pipe, above mentioned, and

into the central line at the gate-vault, or directly into the force-main in the central chamber, back of the check-valve, and are controlled by sluice-gates.

Three 4 by 5-ft. openings, at Elevations 320, 331, and 342, pierce the walls between the north and south chambers, being controlled on the north and south sides by sluice-gates and flap-valves, respectively. The north chambers are connected with the lake through arched openings at the elevations above given, and have stop-log grooves.

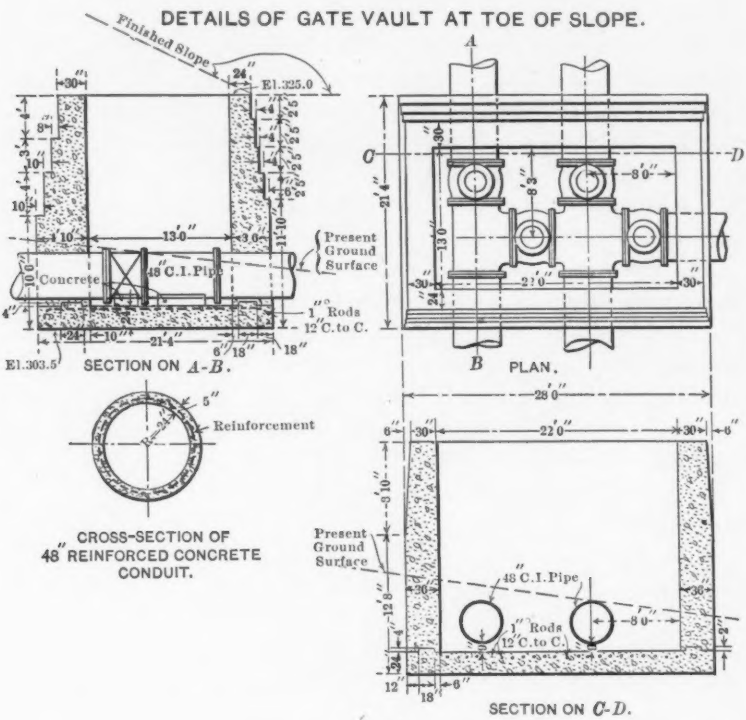


Fig. 2.

Reinforced concrete intakes conduct the water through the dam to the gate-house. At Elevation 350 two overflow openings drain through two lines of 6-in. pipe to gutters on each side of the floor of the central chamber. The central and south side chambers are connected by 12-in. drain pipes, controlled by sluice-gates.

FIG. 1.—RIP-RAP ON DAM. RESERVOIR NEARLY FULL.

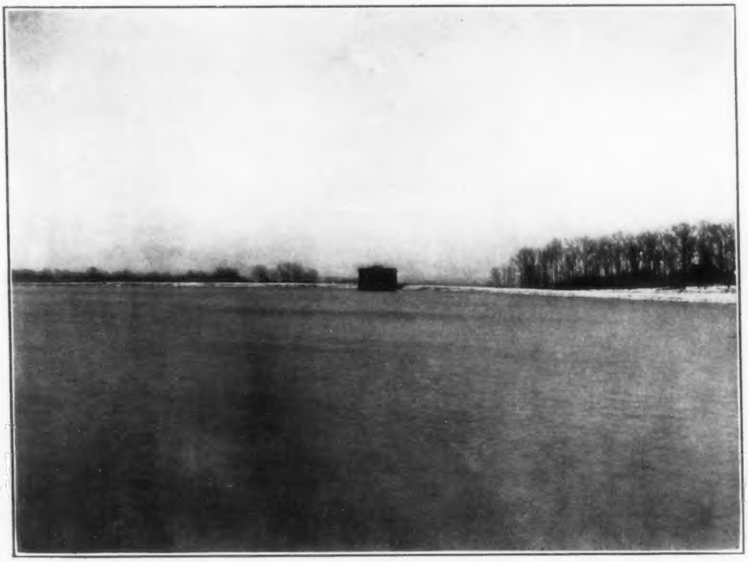


FIG. 2.—LOOKING SOUTHWARD ACROSS BASIN. RESERVOIR FULL.

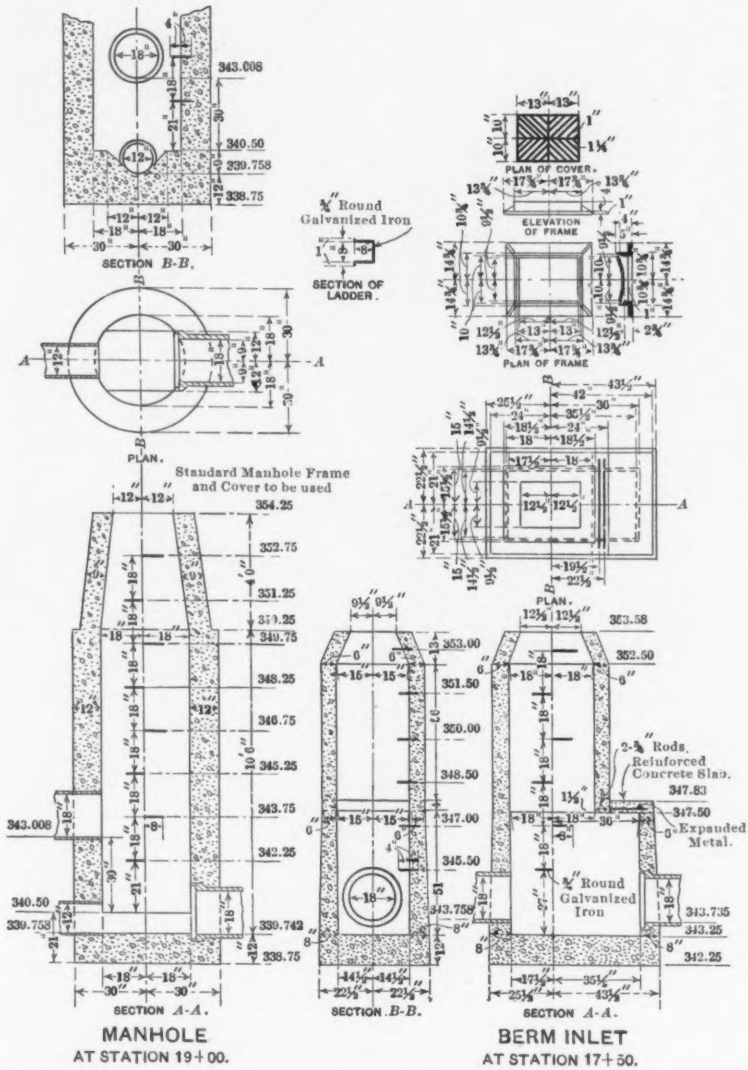

SECTION B-B.
PLAN.
Standard Manhole Frame and Cover to be used
SECTION A-A.
MANHOLE
AT STATION 19+00.
¾" Round Galvanized Iron
SECTION OF LADDER.
PLAN OF COVER.
ELEVATION OF FRAME
PLAN OF FRAME
PLAN.
SECTION B-B.
2-⅜" Rods, Reinforced Concrete Slab.
Expanded Metal.
¾" Round Galvanized Iron
SECTION A-A.
BERM INLET
AT STATION 17+50.

FIG. 3.

TABLE 1.—COST OF LAKE ASH-

Item No.	Items.	Total quantities.	Rate.
1......	Stripping beneath dam, including boulders..................	13 500 cu. yd.	$0.38
2......	Removing top soil....................................	40 000 " "	0.35
3......	Removing clay..	80 000 " "	0.30
4......	Earth excavation from basin........................	273 000 " "	0.20
5......	Loose rock excavation from basin....................	125 000 " "	0.39
6......	Solid rock excavation from basin....................	169 000 " "	0.82
7......	Earth excavation, shallow trench....................	5 360 " "	0.40
8......	Loose rock excavation, shallow trench...............	1 210 " "	0.80
9......	Solid rock excavation, shallow trench...............	410 " "	2.00
10......	Earth excavation, deep trench.......................	14 000 " "	0.50
11......	Loose rock excavation, deep trench..................	6 150 " "	1.10
12......	Solid rock excavation, deep trench..................	260 " "	3.00
13......	Back-fill..	20 090 " "	0.18
14......	Rolled or puddled clay in trench....................	5 000 " "	0.35
15......	Rolled or puddled clay in cut-off wall..............	3 720 " "	0.35
16......	Rolled or puddled clay in dam.......................	35 000 " "	0.35
17......	Rolled selected material in dam.....................	152 000 " "	0.20
18......	Random rock and earth in dam........................	95 000 " "	0.15
19......	Top soil hauled and spread..........................	30 000 " "	0.30
20......	Top soil rolled and seeded..........................	75 000 sq. yd.	0.03
21......	Slope paving, Elevations 340 to 353.................	15 700 " "	1.60
22......	Slope paving, Elevations 330 to 340.................	3 670 " "	1.80
23......	Slope paving, Elevations 320 to 330.................	2 000 " "	2.00
24......	Slope paving, below Elevation 320...................	2 000 " "	2.10
25......	Plain concrete in cut-off wall......................	560 cu. yd.	8.00
26......	Reinforced concrete in cut-off wall.................	2 060 " "	10.00
27......	Plain concrete in bonding wall......................	210 " "	8.00
28......	Tongued and grooved sheeting left in trench.........	100 000 ft. b. m.	50.00
29......	Plain sheeting left in trench.......................	80 000 " " "	40.00
30......	Furnishing and laying 8-in. vitrified pipe..........	500 lin. ft.	0.40
31......	Furnishing and laying 12-in. vitrified pipe.........	500 " "	0.60
32......	Furnishing and laying 18-in. vitrified pipe.........	1 475 " "	1.00
33......	Furnishing and laying 12-in. vitrified tile, including stone or gravel........	1 490 " "	0.60
34......	Storm-water inlets, complete........................	8	55.00
35......	Gate-house, substructure............................	....	....
36......	Gate-vault..	....	....
37......	Reinforced concrete conduit, 48-in..................	1 010 lin. ft.	7.00
38......	Cement grout, 1 : 2.................................	500 bags.	0.75
39......	Neat cement grout...................................	100 "	0.65
	Rock lining in dam..................................	cu. yd.	0.15
	Scraper work } East Drive excavation	" "	0.35
	Loose rock } East Drive excavation..................	" "	0.50
	Solid rock } East Drive excavation	" "	1.00
	Extra yardage outside 2½ : 1 slope..................	" "	0.20
	Excess yardage, inlets..............................	" "	8.00
	Excess material moved on 11th Street................	....	....
	Allowed in west puddle trench to give total excavation..	....	....
	Excess concrete, gate-house intakes.................	cu. yd.	6.50
	Excess reinforced concrete, intakes.................	" "	26.00
	Allowance on Slingluff Avenue fill..................	....	....
	Removing top soil on Park Board's property..........	cu. yd.	0.35
	Allowance on Park Board fill........................	....	....
	Fill, Beech Avenue and 7th Street...................	cu. yd.	0.40
	Dressing, not in original contract..................	....	....
	Miscellaneous force accounts........................	....	....
	Totals..	....	....
	City credits..	....	....

* The increased cost of the deep trench was due to the change in plan of the cut-off

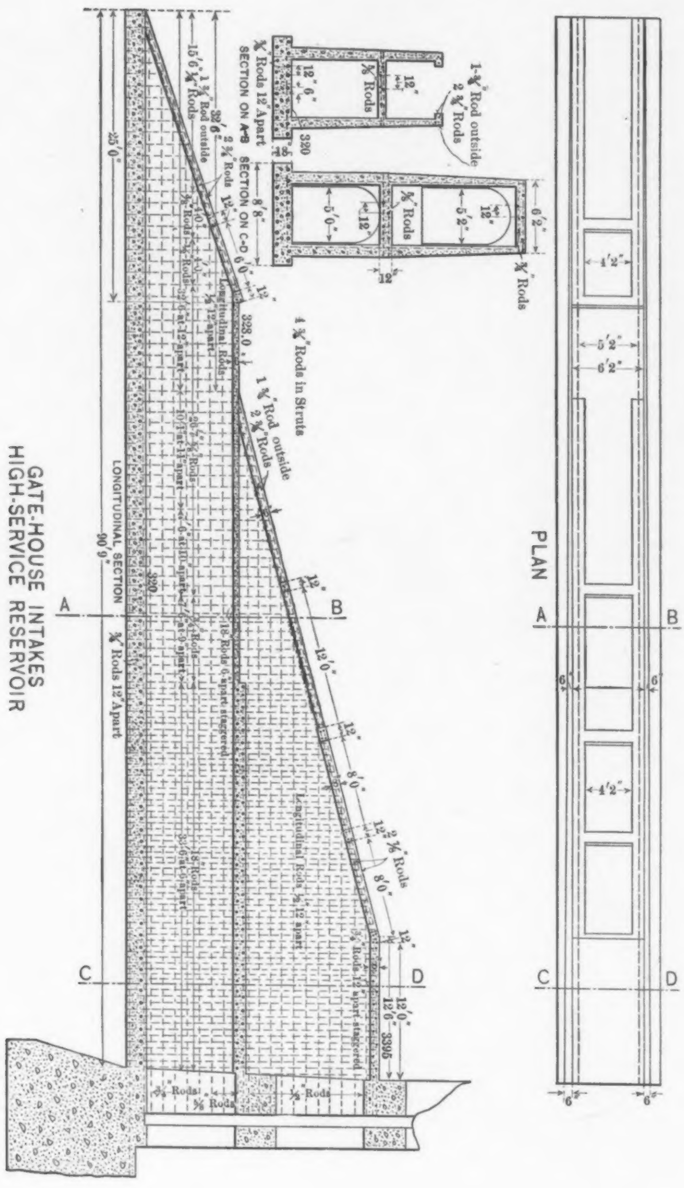
† A saving of $10 000 was made at the dam due to the change in plan. The item $47 289.41 These prices were reduced until $10 000 were saved. The actual saving on the dam, in-mates.

§ This item is made up of three force accounts: Extra work on Liberty Road, $482.90;

BURTON HIGH-SERVICE RESERVOIR.

Estimated cost.	Total quantities.	Actual cost.	Increased cost.	Decreased cost.	Force accounts.	Total actual cost.
$5 139.00	13 978	$5 311.64	$181.64	....	....	$5 311.64
14 000.00	32 886	11 510.10	....	$2 489.90	....	11 510.10
21 000.00	63 846	19 153.80	....	4 846.20	....	19 153.80
54 600.00	161 307	32 261.40	....	22 338.60		
48 750.00	98 752	38 513.28	....	10 236.72	$4 878.63§	340 020.49
138 580.00	322 399	264 367.18	125 787.18	....		
2 144.00	6 314	2 525.60	361.60	....	....	2 525.60
968.00	1 778	1 422.40	454.40	....	....	1 422.40
820.00	1 097	2 194.00	1 374.00	....	....	2 194.00
7 000.00	7 415	3 707.50	....	3 292.50	....	3 707.50
6 765.00	8 078	8 885.80	2 120.70	....	....	8 885.80
780.00	9 418	28 254.00	27 474.00*	....	....	28 254.00
3 616.20	2 725	490.50	....	5 125.70	....	490.50
1 750.00	3 910	1 368.50	....	381.50	....	1 368.50
1 302 00	25 082	8 761.20	7 459.70	....	1 295.74	10 056.94
12 250.00	37 784	13 224.40	974.40	....		
30 400.00	199 226	21 845.20	....	8 554.80	1 649.26	47 289.41†
14 250.00	137 137	20 570.55	6 320.55	....		
9 000.00	20 805	6 241.50	....	2 758.50	....	6 241.50
2 250.00	72 603	2 178.09	....	71.91	....	2 178.09
25 120.00	....	....	....	16 295.73	8 824.27	8 824.27
6 606.00	....	....	....	....	....	....
4 000.00	....	....	....	....	....	....
4 200.00	....	....	....	....	....	....
4 480.00	60.81	486.48	....	3 993.52	....	486.48
20 000.00	....	....	....	20 600.00	....	....
1 680.00	....	....	....	1 680.00	....	....
5 000.00	....	....	....	5 000.00	....	....
3 200.00	384 052	15 362.08	12 162.08	....	....	15 362.08
200.00	450	180.00	....	20.00	....	180.00
300.00	546.04	327.62	27.62	....	....	327.62
1 475.00	1 398.44	1 398.44	....	76.56	....	1 398.44
894.00	1 384	830.40	....	63.60	....	830.40
440.00‡	....	540.00	100.00	....	....	540.00
28 600.00	....	29 130.95	530.95	....	1 306.72	30 437.67
2 150.00	....	2 150.00	....	....	582.87	2 732.87
7 070.00	1 010	7 070.00	....	....	337.64	7 407.64
375.00	....	....	....	375.00	....	....
65.00	....	....	....	65.00	....	....
....	36 448	5 467.20	5 467.20	....	....	5 467.20
....	1 145	400.75	400.75	....	....	400.75
....	50	25.00	25.00	....	....	25.00
....	175	175.00	175.00	....	....	175.00
....	1 200	240.00	240.00	....	....	240.00
....	4.4	35.20	35.20	....	....	35.20
....	....	100.00	100.00	....	....	100.00
....	....	198.90	....	....	....	198.90
....	30.2	196.30	196.30	....	....	196.30
....	47.7	1 240.20	1 240.20	....	....	1 240.20
....	....	1 200.34	1 200.34	....	....	1 200.34
....	4 988	1 745.80	....	....	....	1 745.80
....	....	2 500.00	2 500.00	....	....	2 500.00
....	2 119	847.60	....	....	....	847.60
....	....	....	....	....	1 286.81	1 286.81
....	....	....	....	....	3 734.77	3 734.77
$494 810.20	....	....	....	....	$23 896.71	$518 531.61
....	....	....	....	....	....	217.69
				Net total....	....	$518 313.92

wall trench, and to the fact that more rock was found than was estimated.
should be $57 289.41 at the original prices for depositing the different materials in the dam.
cluding the force accounts and the rock lining, was $4 143.39 less than the original esti-
‡ A manhole was built which was not in the original contract.
sewer extension, $2 531.03; and drainage, springs, etc., $1 864.70.



GATE-HOUSE INTAKES
HIGH-SERVICE RESERVOIR

Fig. 4.

Under normal conditions, the reservoir will act as an equalizer between the pumps at the Mt. Royal pumping station and the main on North Avenue, which supplies a large territory. Any surplus pumped, greater than the consumption, passes through the check-valve to the basin at the west end of the reservoir, the flap-valves in the side chambers preventing any discharge into the reservoir through these chambers and the intakes. If the pumpage is less than the consumption, or if the pumps are shut down, water is drawn through the side chambers, the check-valve preventing its return by way of the force-main. By this arrangement, circulation is obtained in the reservoir, as all water enters at the west end through the force-main and is drawn out through the intakes at the east end.

The excavated portion of the basin was hand-paved between Elevations 340 and 353 with rock from the shovels. On the dam, after the rock lining was dressed, rip-rap was laid as an additional protection to the slope.

In December, 1910, chloride of lime having been used freely, about 5 ft. of water were pumped into the basin and drawn off. On December 16th pumping recommenced, and was continued until January 11th, 1911, when the water surface reached Elevation 350.

As far as it is possible to determine, no water has passed through the dam, the insignificant quantity of seepage observed to date seeming to come entirely through the natural ground, and, with the exception of the springs led from the dam foundations through blind drains, being considerably removed from the toe of the dam.

The cost of the work is given in Table 1.

The work was designed by Emory Sudler, Assoc. M. Am. Soc. C. E., Assistant to Alfred M. Quick, M. Am. Soc. C. E., Water Engineer, and was executed under his supervision.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

THE FLOOD OF MARCH 22D, 1912, AT PITTSBURGH, PA.*

BY KENNETH C. GRANT, ASSOC. M. AM. SOC. C. E.

TO BE PRESENTED NOVEMBER 6TH, 1912.

On March 22d, 1912, the Allegheny and Monongahela Rivers, which join at Pittsburgh to form the Ohio, reached a gauge height of 28.1 ft., the highest stage since February 16th, 1908, when there was a 30.7-ft. flood. Except for a second flood, on March 20th, 1908, which reached a stage of 27.3 ft., no other flood of importance had occurred at Pittsburgh for four years, although the danger line, 22 ft., had been reached several times, the highest stage occurring on January 31st, 1911, when the rivers rose to a gauge height of 25.2 ft. The record stage of 35.5 ft. occurred on March 15th, 1907, when about $6 000 000 of flood damage resulted.

In view of the exhaustive studies looking toward methods of flood relief that have been made by the Flood Commission of Pittsburgh, which has recently published its report, it is thought that a brief description of the recent flood will prove of interest. To this description has been added a graphical study of the peak reduction that could have been obtained if the system of storage reservoirs proposed by the Flood Commission had been in operation.

NOTE.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

* The portion of this paper describing the flood was in large part written directly after its occurrence, and published in *Engineering News* of April 4th, 1912. Since this first writing, additional data have become available, and the description has been modified and enlarged accordingly.

At 8 A. M., on March 21st, 1912, the gauge at Pittsburgh registered 17.9 ft. For 5 days prior to this date the rivers had been flowing fairly full, the gauge readings at Pittsburgh varying between 13 and 18 ft. The weather had been mild, and the high temperatures that prevailed for part of the time had gradually brought the ice out of the tributaries and head-waters of the Allegheny. The ice on the Monongahela and its tributaries had gone out some time before. Had cold weather continued until the 20th, when the flood rainfall began, the ice on the Allegheny would have been held back until then, the flood would have been considerably higher, and might have been dangerously increased by ice gorges. The ice moved out of the Upper Allegheny and its tributaries on the 17th. On the Clarion River a heavy gorge formed below Clarion, raising the stage at that place to 13.0 ft., on the 16th, which rapidly reduced to 4.6 ft. on the 17th, when the ice moved out early in the morning. Ice was running on the Allegheny at Parker on the 16th and 17th, and began passing Freeport on the morning of the 17th.

During the 21st the rivers at Pittsburgh rose rapidly and crossed the danger line, or the 22-ft. stage, about 2 P. M., rising steadily until 5 A. M. on the 22d, when the maximum of 28.1 ft. was reached. The crest remained at this level for 2 hours and then dropped at about the same rate as it had risen, falling below the danger line at about 2 A. M. on the 23d, and registering 19.6 ft. at 8 A. M. The rise was probably checked to some extent by a sudden drop in temperature on the 21st. The water was above the danger line for about 36 hours, during which time the total discharge above the 22-ft. stage was about 6 221 000 000 cu. ft. The discharge at the crest of 28.1 ft. was about 300 000 sec-ft.

All the rain causing this flood fell practically in the 12 to 14 hours preceding 8 A. M on the 21st, although in some sections it continued for a few hours after this time. After the middle of March, with the exception of an inch or two in the extreme northern part of the Allegheny Basin, there was no snow on the ground to contribute to the flood run-off.

The precipitation was lightest over the Northern Allegheny Basin, where it was mostly in the form of snow. The Upper Allegheny, therefore, contributed little to the rise, especially as practically all the 6 in. of snow which fell was still on the ground on March 23d. French Creek was at a fairly high stage, being above 10 ft. throughout the

20th and 21st. The Clarion, however, rose to only 6.6 ft., 9.4 ft. below record stage, and Red Bank, Mahoning, and Crooked Creeks were not large contributors, so that the Allegheny at Kittanning rose to only 17.2 ft., 12 ft. below record stage, at 4 P. M. on the 21st. At Freeport, the river registered 23.2 ft. at 6 P. M. on the 21st, when the water was rising at the rate of 0.1 ft. per hour. The main contributor from the Allegheny Basin was the Kiskiminetas River, which reached a stage of 28.0 ft. at Avonmore at 8 P. M. on the 21st, 9 hours before the crest was reached at Pittsburgh.

On the Monongahela Basin, the Cheat, Tygart Valley, and West Fork Rivers were not important factors in the rise. The Youghiogheny, however, was an important contributor, and, with the Kiskiminetas, was the main cause of the high stage reached at Pittsburgh. It reached a stage of 22.7 ft. at West Newton at 8 P. M. on the 21st. Except during the flood of March, 1907, when the maximum at West Newton was 28.2 ft., this river had never been higher than 22.0 ft. at this station. The time necessary for floods of these heights to move from West Newton and from Avonmore to Pittsburgh is from 9 to 12 hours, and both the Youghiogheny and Kiskiminetas, therefore, must have delivered their greatest discharge at Pittsburgh at about flood-peak time, between 5 and 7 A. M. on the 22d.

The high stages in the Kiskiminetas and Youghiogheny Rivers were to be expected, as the rainfall was heaviest over their basins. The maximum rainfall recorded during this storm was at Lycippus, Pa., on the Kiskiminetas Basin, where 3.05 in. fell in the 12 to 14 hours following the evening of March 20th. The next greatest rainfall was at Johnstown, Pa., on the same basin, where 2.91 in. fell during the same period. The region of greatest rainfall centered around these stations. Toward the north the recorded rainfall decreases rapidly in amount, and it is evident that, with the exception of the Kiskiminetas Basin, little of the Allegheny Basin received more than 1 in. of rainfall, while a considerable portion received only ½ in., or less. Toward the south the amount of rainfall recorded decreases more gradually to about 1 in. near Grafton and Parsons, W. Va., and farther south, near the head-waters of the West Fork, to less than ½ in. A considerable portion of the Monongahela Basin, therefore, mostly included within the Youghiogheny water-shed, received 2 in. or more of rainfall, while the larger part received more than 1 in.

The rainfall causing this flood is given in Table 1, and the map of the basins, Fig. 1, shows its distribution. Table 2 gives the daily gauge heights at stations on the main rivers and their tributaries before and after the time of the Pittsburgh peak, including time and height of maximum stage in so far as records exist.

TABLE 1.—RAINFALL CAUSING FLOOD OF MARCH 22D, 1912, AT PITTSBURGH.*

ALLEGHENY BASIN.		MONONGAHELA BASIN.		OHIO BASIN.‡	
Station.	Depth, in inches.	Station.	Depth, in inches.	Ohio Basin.	Depth. in inches.
Allegany, N. Y.	0.65	Buckhannon, W. Va.	0.30	Beaver Dam, Pa.	0.98
Angelica, N. Y.	0.92	California, Pa.	1.65	Claysville, Pa.	0.68
Baldwin, Pa.	0.64	Central Station, W. Va.	0.70	Davis Island Dam, Pa.	0.93
Bolivar, N. Y.	1.00	Confluence, Pa.	2.37	Greenville, Pa.	0.98
Brookville, Pa.	0.35	Creston, W. Va.	0.43	Grove City, Pa.	0.93
Clarion, Pa.	0.12	Elkins, W. Va.	0.68	Sharon, Pa.	0.82
Derry, Pa.	1.99	Fairmont, W. Va.	1.59	Ellwood City, Pa.	0.91
Franklin, Pa.	1.07	† Glenville, W. Va.	0.45	Beaver Falls, Pa.	0.96
Freeport, Pa.	1.61	Grafton, W. Va.	0.80	Coraopolis, Pa.	0.86
Haskinville, N. Y.	0.50	Greensboro, Pa.	1.54		
Herr Island Dam, Pa.	1.15	Greensburg, Pa.	2.53		
Hunt, N. Y.	0.40	Irwin, Pa.	1.64		
Indiana, Pa.	0.20	Lock No. 4, Pa.	1.19		
Johnstown, Pa.	2.91	Lost Creek, W. Va.	0.73		
Lycippus, Pa.	3.05	Mannington, W. Va.	1.50		
Olean, N. Y.	1.00	Morgantown, W. Va.	2.00		
Otto, N. Y.	0.40	Parsons, W. Va.	1.00		
Parker, Pa.	0.79	Philippi, W. Va.	0.71		
Pittsburgh, Pa.	0.58	Pickens, W. Va.	0.69		
Saegerstown, Pa.	0.85	Rowlesburg, W. Va.	1.56		
Saltsburg, Pa.	1.63	* Smithfield	1.47		
Springdale	1.32	Somerset, Pa.	1.55		
* Volusia, N. Y.	0.50	Uniontown, Pa.	1.65		
Warren, Pa.	0.66	West Newton, Pa.	2.19		
Westfield, N. Y.	0.39				

* This rain fell in the 12 to 14 hours preceding 8 A. M. on March 21st; in a few sections it continued for a few hours after this time.

† Just outside of basin.

‡ Records used in locating isohyetal lines on Fig. 1.

Damage by Flood.—The damage caused by this flood in Pittsburgh and in other communities bordering the rivers above and below that city was considerable. A careful census of flood damage, made by the Pittsburgh Flood Commission after the 27.3-ft. flood of March 20th, 1908, showed a direct loss at Pittsburgh alone, for a flood about 1 ft. lower than the recent stage, of more than $400 000. It would seem reasonable, therefore, to place the direct loss in the recent flood at about $500 000, although, when complete information is at hand, this figure may be modified, especially as certain streets in the flooded district were raised during 1911. About 250 acres of the low-lying

TABLE 2.—DAILY GAUGE HEIGHTS AT RIVER STATIONS, FLOOD OF MARCH 22D, 1912, AT PITTSBURGH.

Stream.	Station.	MARCH.											HIGHEST RECORDED STAGE.	
		15th	16th	17th	18th	19th	20th	21st	22d	23d	24th	25th	Stage.	Date.
Allegheny River	Red House	5.2	7.8	7.0	7.8	8.4	a 9.4	8.7	7.3	....	....	....	13.6	Mar. 2, 1910
Allegheny River	Warren	F 1.8	F 3.0	5.5	7.0	7.7	8.9	9.7	7.7	6.0	....	....	17.4	Mar. 17, 1865
Tionesta Creek	Nebraska	3.7	5.5	3.4	4.4	4.5	4.7	4.0	3.6	....	....	....	....	....
Oil Creek	Rouseville	2.3	3.3	3.6	4.0	3.8	4.5	3.6	....	....	....	....	....	....
No. Br. French Creek	Kimmeytown	3.7	5.5	6.6	8.9	6.7	8.5	6.0	4.3	3.5	....	....	13.9	....
Cussewego Creek	near Meadville	5.0	....	9.7	10.5	10.4	10.3	9.9	9.4	8.5	....	....	....	....
Sugar Creek	Wyattville	4.0	3.4	3.7	3.9	3.8	3.6	3.7	3.7	3.6	....	....	....	....
French Creek	Carlton	5.5	8.2	6.9	7.8	9.2	10.5	b 10.6	....	....	....	....	16.5	1888
Allegheny River	Franklin	F 2.3	F 4.7	5.9	9.1	9.8	11.6	11.6	9.9	7.9	....	....	....	Mar. 17, 1865
Clarion River	Clarion	F 2.7	c 13.0	4.6	6.0	4.3	6.0	6.6	5.6	5.0	....	....	16.0	Mar. 20, 1905
Allegheny River	Parker	F 2.7	F 5.5	7.6	10.4	10.7	11.9	12.7	11.4	8.4	....	....	28.0	Mar. 1865
Allegheny River	Kittanning	....	9.5	16.4	13.7	14.1	15.5	d 17.2	14.9	....	....	....	29.3	Mar. 1865
Red Bank Creek	St. Charles	7.2	6.2	5.9	5.5	5.7	5.8	6.2	5.7	....	....	....	....	....
Mahoning Creek	Furnace Bridge	7.5	6.6	5.5	5.5	5.2	5.6	e 6.4	7.2	....	....	....	....	....
Crooked Creek	Hileman's Farm	5.6	3.2	3.8	3.3	2.8	2.4	f 6.5	5.3	3.6	....	....	....	....
Black Lick Creek	Black Lick	7.2	6.9	5.4	5.5	5.4	5.6	g 10.1	8.0	....	....	....	13.2	Mar. 14, 1907
Conemaugh River	Johnstown	6.7	7.0	4.5	5.9	5.9	7.3	h 11.8	10.0	6.3	....	....	21.0	May 31, 1889
Kiskiminetas River	Saltsburg	3.1	8.5	4.5	4.3	4.3	4.3	i 8.5	12.0	5.8	....	....	22.1	1859
Kiskiminetas River	Avonmore	11.7	15.6	10.8	10.6	10.4	11.7	j 22.8	22.0	....	....	....	34.9	1859
Allegheny River	Freeport	8.0	14.0	14.4	12.3	14.7	15.5	k 19.3	22.2	15.5	....	....	32.7	Feb. 18, 1891
West Fork River	Enterprise	11.8	12.2	9.0	6.3	4.0	4.1	8.9	7.6	....	....	....	33.0	July 1888
Tygart Valley River	Belington	6.3	12.2	8.4	6.0	5.1	5.4	6.7	7.9	....	....	....	19.6	July 1888
Tygart Valley River	Fetterman	12.0	14.8	11.5	8.1	6.7	8.8	10.3	11.2	9.2	10.3	10.8	29.0	July 1888
Cheat River	Rowlesburg	5.6	9.0	5.6	4.0	3.7	4.4	l 6.0	6.7	5.2	....	....	22.0	July 10, 1888
Cheat River	Ices Ferry	7.8	9.3	6.3	5.4	5.0	6.2	m 9.5	8.3	6.3	....	....	....	July 10, 1888
Monongahela River	Fairmont	19.2	23.6	22.0	17.6	16.6	16.6	n 19.3	20.6	19.0	....	....	37.0	July 10, 1888
Monongahela River	Greensboro	12.8	20.9	15.3	11.8	10.3	10.5	o 16.3	18.3	13.9	....	....	39.0	July 10, 1888
Monongahela River	Lock No. 4	17.5	25.7	23.2	17.0	13.3	12.7	p 18.3	27.5	20.5	....	....	42.0	July 11, 1888
Youghiogheny River	above Confluence	10.9	9.6	6.1	5.5	5.3	7.2	15.3	10.2	6.5	....	....	16.8	Mar. 14, 1907
Youghiogheny River	below Confluence	4.4	8.3	5.0	4.4	4.3	5.4	9.8	10.3	6.0	....	....	18.6	Mar. 14, 1907
Casselman River	Confluence	10.5	8.8	5.1	4.9	4.7	7.0	17.0	9.6	5.6	....	....	18.1	Mar. 14, 1907
Laurel Hill Creek	Confluence	8.0	7.0	4.1	4.0	3.8	5.2	14.4	7.2	4.1	....	....	17.0	Mar. 14, 1907
Youghiogheny River	Connellsville	8.2	10.9	7.0	6.6	6.1	7.6	q 14.7	12.3	....	....	....	....	Mar. 14, 1907
Youghiogheny River	West Newton	4.9	14.1	8.3	6.0	5.3	5.8	r 11.8	19.8	9.9	....	....	28.2	Mar. 14, 1907
Ohio River	Pittsburgh	9.6	18.3	17.9	13.1	13.4	13.3	17.9	s 28.1	19.6	....	....	35.5	Mar. 15, 1907
Ohio River	Wheeling	12.8	19.7	27.0	25.5	21.4	21.0	21.9	t 33.3	38.4	31.4	25.5	53.1	Feb. 7, 1884
Ohio River	Parkersburg	15.7	23.4	26.2	28.0	26.0	23.6	24.0	29.0	34.8	u 36.5	35.8	53.9	Feb. 9, 1884
Ohio River	Cincinnati	37.2	42.6	44.0	45.4	46.6	47.0	46.5	47.0	45.2	45.8	v 50.2	71.1	Feb. 14, 1884
Ohio River	Cairo	34.4	34.9	....	38.2	40.4	42.4	44.1	45.4	46.4	....	48.5	w 54.0	Apr. 6, 1912

F. Frozen. *a*. Max. at noon, 9.65. *b*. Max. at noon, 10.7. *c*. Ice gorge; broke at 5 A. M. on 17th. *d*. Max. at 4 P. M., 17.2. *e*. Max. at 5 P. M., 8.4. *f*. Max. at 5 P. M., 9.4. *g*. Max. at 1 P. M., 11.1. *h*. Max. at noon, 13.6. *i*. 14.0 at 2 P. M.; still rising. *j*. Max. at 8 P. M., 28.0. *k*. 23.2 at 6 P. M.; rising 0.1 ft. per hour. *l*. Max at 2 P. M., 7.0. *m*. Max. at 5 P. M., 9.8. *n*. Max. at 6 P. M., 21.4. *o*. Max at 4 P. M., 20.0. *p*. 26.2 at 8 P. M.; rising 0.5 ft. per hour. *q*. Max. at 3.30 P. M., 17.35. *r*. Max. at 8 P. M., 22.7. *s*. Max. 5 to 7 A. M., 28.1. *t*. Max. at midnight, 38.6 *u*. Max. at 2 P. M., 36.7. *v*. Mar. 26, 8 A. M., 52.2; Mar. 27, 8 A. M., 53.2, 3 P. M., 53.4 (max.); Mar. 28, 8 A. M., 52.8 and falling. *w*. Previous max. 52.2, in Feb., 1883.

portions of the city, including perhaps 40 acres of railroad and industrial yards, were submerged to a maximum depth of about 9 ft. Several miles of main-line railroad and of street-car tracks were submerged, the water reaching a depth of about 8 ft. on the Baltimore and Ohio tracks along the right bank of the Allegheny. Approximately 6 miles of streets and alleys were submerged to a greatest depth of about 7 ft.

The night of March 21st was one of discomfort to thousands of people living in the low-lying portions of the city. Most of the residents of these districts moved their furniture upstairs and remained in their houses. Others left their homes in boats manned by the police. Dozens of small boats plied the streets of Lower Allegheny, carrying people to and from their homes. Damage and discomfort of a similar nature were experienced in all the low-lying towns along the rivers above and below Pittsburgh.

The interference with business was not serious. The down-town business section was not invaded, as in the great flood of 1907, although cellars were flooded by seepage and back-water from the sewers, and pumps had to be operated while the rivers were at flood stage. Merchandise and other materials stored in cellars and basements had to be removed to the sidewalks or to higher portions of the buildings. The merchants in the lower districts, as a precaution, had stop-plank floodgates in place on the night of the 21st. The accurate prediction of 28.0 ft., made by the Local Forecaster of the United States Weather Bureau, gave ample time to prepare for the flood, and much valuable property was saved thereby.

The following details gathered from the newspaper accounts of the flood damage, have not been verified, but are believed to be reasonably accurate.

The interruption to traffic caused considerable inconvenience and loss of time on March 21st and 22d. In some sections street-car service was seriously interfered with by flooded tracks, and thousands were late for work on the 22d on account of re-routeing and transferring. The Baltimore and Ohio Railroad tracks and passenger station on the right bank of the Allegheny River were flooded by noon on the 21st, and all trains were transferred to the Smithfield Street Station. On the South Side, or left bank of the Monongahela, some of the tracks of the Pittsburgh and Lake Erie Railroad were under a

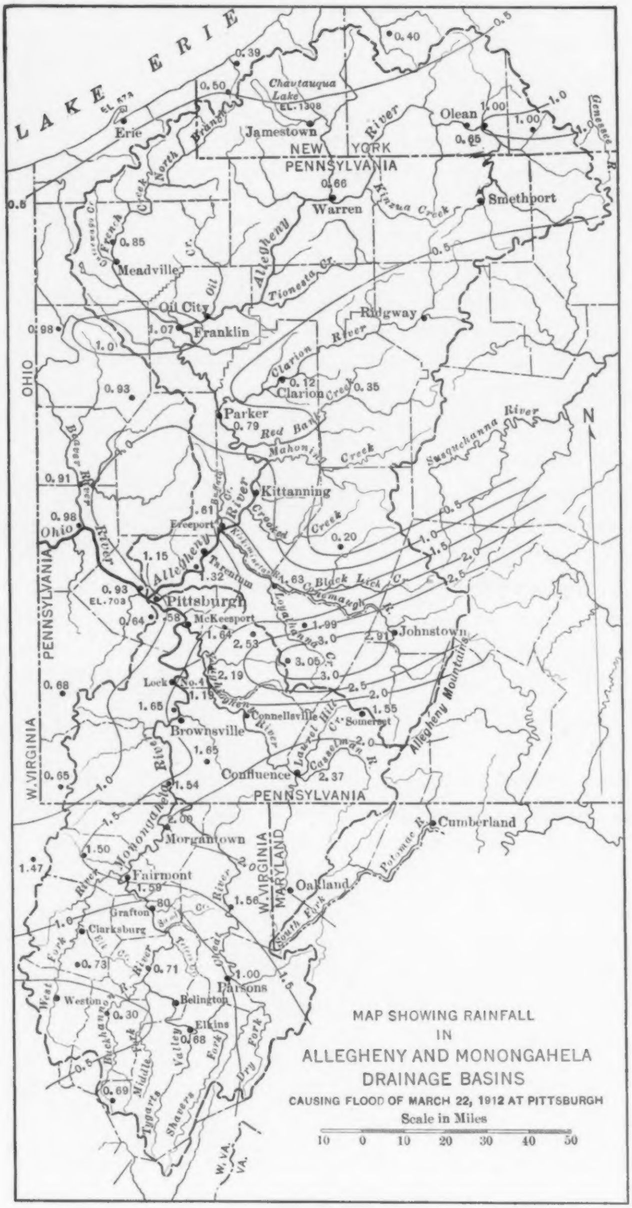

MAP SHOWING RAINFALL
IN
ALLEGHENY AND MONONGAHELA
DRAINAGE BASINS
CAUSING FLOOD OF MARCH 22, 1912 AT PITTSBURGH
Scale in Miles
10 0 10 20 30 40 50
LAKE ERIE
Erie
Jamestown
Olean
Smethport
Warren
Meadville
Oil City
Franklin
Ridgway
Clarion
Parker
Kittanning
Freeport
Pittsburgh
McKeesport
Johnstown
Brownsville
Connellsville
Somerset
Confluence
Cumberland
Morgantown
Fairmont
Oakland
Grafton
Clarksburg
Parsons
Weston
Belington
Elkins
NEW YORK
PENNSYLVANIA
OHIO
W. VIRGINIA
MARYLAND

FIG. 1.

few inches of water, but traffic was not interrupted. At West Newton, on the Youghiogheny River, the Baltimore and Ohio tracks were flooded and the trains were run on the tracks of the Pittsburgh and Lake Erie, on the other bank. Portions of the tracks of the Pennsylvania Railroad along the Kiskiminetas River, between Saltsburg and Salina, and along the Conemaugh River, at Sloan's Cut, east of Blairsville, were washed away.

Several thousand men were laid off on account of the high water entering industrial plants and causing their shut-down. At least 2 000 of these employees, it was stated, would be out of work for a week. The Pennsylvania Department of the National Tube Works, in Second Avenue, shut down for a week when water entered the plant at about 2 A. M. on the 22d. The rolling mill at the Continental Department of the same Company also shut down for a week. About 1 700 men are employed in these two departments. The plants of the A. M. Byers Company, at South Sixth Street, and of the Dilworth, Porter Company, at South Fourth Street, closed down on the night of the 21st, on account of the high water backing into the fly-wheel pits. The Carbon Steel Company and the American Steel and Wire Company's plant at 15th Street had to shut down on the morning of the 22d. The Open-Hearth Department of the Jones and Laughlin Steel Company, at South 27th Street, was shut down on the night of the 21st, as it was feared that the furnaces might be flooded, but, as the water did not reach them, work was resumed on the morning of the 22d. At the mills of the Spang, Chalfant Company, at Etna, all movable machinery and material were transferred to points above flood level. This was also done at the Westinghouse Air-Brake Works, at Wilmerding, on Turtle Creek, a small tributary of the Monongahela near Pittsburgh. This plant closed down on the morning of the 21st on account of high water in Turtle Creek, and in the afternoon the water was 20 in. deep on the floor of the works. Men were engaged all night in cleaning up, and work was resumed on the morning of the 22d. At Hays, on the left bank of the Monongahela, a short distance above Pittsburgh, the Monongahela Iron and Steel Company and the National Car Wheel Works were shut down on the night of the 21st, because of flooded furnaces, throwing about 650 men out of work.

Movement of the Flood Down the Ohio River.—The flood crest reached Rochester, Pa., 25 miles down the Ohio from Pittsburgh, at

PLATE LXXVI.
PAPERS, AM. SOC. C. E.
AUGUST, 1912.
GRANT ON
PITTSBURGH FLOOD.

FIG. 1.—WEST RELIANCE STREET, ALLEGHENY, ON THE MORNING OF MARCH 22D, 1912.

FIG. 2.—RIVER AVENUE, ON THE RIGHT BANK OF THE ALLEGHENY RIVER, ON THE MORNING OF MARCH 22D, 1912.

about 11 A. M. on the 22d, and the streets and cellars in the lower section of the town were flooded. At Steubenville, Ohio, 68 miles below Pittsburgh, the highest stage, 37.0 ft., was reached on the evening of the 22d. Several industrial plants were shut down, and railroad and street-car traffic was interrupted. At Wheeling, W. Va., 90 miles below Pittsburgh, the water rose above the danger line of 36.0 ft. at 1 P. M. on the 22d, and reached the maximum for this flood, 38.6 ft., at about midnight on the 22d. Street-car traffic was interfered with and, on certain lines, was suspended. Basements and cellars in the low districts were flooded, and merchandise had to be removed to the sidewalks and to upper stories. Several mills were obliged to close. The crest reached Parkersburg, W. Va., 183 miles below Pittsburgh, at 4 P. M., on the 24th, when a stage of 37.0 ft., or about 1 ft. above the danger line, was recorded. Little damage is done at this stage at Parkersburg. Cincinnati, Ohio, 468 miles below Pittsburgh, had its highest water, 53.4 ft., or 3.4 ft. above the danger line, at 3 P. M. on March 27th. The flood stage of 50 ft. was reached on the morning of the 25th, and residents in the low-lying districts began to move out of their houses. The water began to overflow some of the railroad tracks on the 26th, when the gauge read 52.2 ft. at 8 A. M., but the service was not interrupted. At Cincinnati a stage of 54 ft. or more prevents trains from entering the Union Central Depot. The river at Cairo, Ill., at the mouth of the Ohio, 967 miles below Pittsburgh, reached its maximum of 54.0 ft. (9 ft. above the danger line) on April 6th. The previous recorded maximum was 52.2 ft., in February, 1883. The river at this point was above the danger line of 45 ft. for 32 days, from March 22d to April 22d, inclusive.

The flood on the Mississippi during April, 1912, was one of the worst ever experienced on that river, and in some sections exceeded all previous records. Many lives were lost, and enormous damage resulted from overflow. The Ohio River was the principal contributor to this rise, although part of the flood-water came from the Lower Missouri and Upper Mississippi. The Arkansas and Red Rivers were not important contributors. Professor H. C. Frankenfield has published* an excellent article recording the history of the conditions preceding this great flood, and the stages of its progress down the river. The data presented in that article bring out the fact that, unquestionably,

* *Engineering News*, April 18th, 1912.

flood stages in the Mississippi are raised by the levees, and that the end has not yet been reached, as far as extreme high stages are concerned, especially in the event of the possible combination of great floods in both the eastern and western tributaries.

Estimated Reduction of the Pittsburgh Crest by Storage Reservoirs. —Before proceeding to a discussion of the reduction of the flood of March 22d, 1912, which could probably have been obtained at Pittsburgh, if a system of flood-control reservoirs had been in operation, the following outline of the studies made along these lines by the Flood Commission of Pittsburgh will perhaps be of interest.

The Report of the Flood Commission was issued in April, 1912. It gives a detailed account of the extensive surveys and studies made by the Commission for the purpose of determining the feasibility of preventing floods at Pittsburgh by storage reservoirs on the Allegheny and Monongahela Basins. The forty-three most favorable sites were selected from a large number considered, and detailed estimates were made of the capacities and costs of the respective reservoirs.

These surveys and studies showed that, at certain sites, the maximum storage feasible was greater than needed for flood control. For example, if all forty-three reservoirs were constructed to maximum capacity, the storage in excess of that required for flood-control purposes would amount to 4 357 000 000 cu. ft. on the Allegheny Basin, and 17 937 500 000 cu. ft. on the Monongahela Basin, the total maximum capacity found available being nearly 103 000 000 000 cu. ft.

The capacities of these reservoirs were cut down, therefore, in each case to the amount necessary to control floods from the drainage areas above the respective dams. The total capacity of the twenty-one projects on the Allegheny Basin then amounted to 49 725 800 000 cu. ft., and of the twenty-two on the Monongahela Basin to 30 772 000 000 cu. ft., or a total of 80 497 800 000 cu. ft. These reservoirs would control 8 454 sq. miles, or 73% of the Allegheny drainage area, and 3 379 sq. miles, or 46% of the Monongahela drainage area; or a total of 11 833 sq. miles, or 62% of the total drainage area above Pittsburgh. The total cost of the forty-three projects would be $34 170 800.

An analysis of the effect of these forty-three reservoir projects in reducing floods at Pittsburgh was then made for the eleven principal floods from 1898 to 1908. Prior to 1898, sufficient data for such studies were not available.

MAHONING CREEK, PENNSYLVANIA. VIEW UP STREAM AT DAM SITE.

This study of the Pittsburgh peak reduction was made graphically by a series of mass diagrams, in which the estimated flood volumes contributed by the various controlled tributaries were assembled in their calculated positions under the main volume curve representing the Pittsburgh peak, or flood crest, and the reduced peak was obtained by deducting these tributary volumes from the volume of the main peak, which is what would actually take place if the proposed reservoirs on these tributaries were constructed. A diagram of this kind was constructed for each of the eleven floods considered.

By these diagrams it was ascertained that, of the eleven floods studied, all except the highest, that of March, 1907 (35.5. ft.), would have been reduced well below the danger line. The March, 1907, flood would have been reduced to 25.3 ft., and would have been above 22 ft. for 22 hours, as compared with the actual 61 hours. These eleven floods remained above the 22-ft. stage from 34 to 86 hours. The longest flood, that of March, 1905, which reached the gauge height of 29 ft., remained above the 22-ft. stage for 86 hours. This flood would have been reduced to a gauge height of 18.3 ft. It was also found that, with 62% of the total drainage area controlled, a reduction of from 39 to 84% of the peak discharge would have been obtained, or an average of 53 per cent.

Relative Effectiveness of Reservoirs.—It is evident that the amount of flood reduction effected by a given reservoir project is not merely a matter of the magnitude of the flood in the tributary it controls; it is just as much a question of how the contribution of that tributary arrives, with reference to the time of the crest, in the main river.

Inspection of these mass diagrams, by which the reductions of the Pittsburgh peaks were estimated, showed that certain tributaries are notable and repeated offenders in Pittsburgh floods, that is, their flood-volume curves arrive there at about the same time as the Pittsburgh peak. It is obvious that the most effective storage would be that controlling their flood-waters. Therefore, a study of the relative effectiveness of the various projects was made on this basis, and it was found that, except in the case of the 1907 flood, a comparatively small number of reservoir projects is needed to reduce floods at Pittsburgh to below the danger line; and that certain projects would have but little effect, and, therefore, should be dismissed from further consideration for flood-prevention purposes.

Thus, it was found that if fifteen of these least effective projects were rejected, reducing the cost by about $6 000 000, all the eleven floods considered would still be reduced to below the danger line except that of 1907, which would be reduced to 26.1 ft., or only 0.8 ft. above the point to which it would be reduced by the construction of all forty-three projects.

A further analysis, taking into account the relative cost as well as the relative effectiveness of the respective projects, reduced the number to seventeen. These seventeen selected projects control a drainage area of 8 023 sq. miles, or 69% of the Allegheny Basin, and 2 159 sq. miles, or 29.5% of the Monongahela Basin; or a total of 10 182 sq. miles, or 54% of the total drainage area above Pittsburgh. Thirteen of the projects, with a total capacity of 42 178 500 000 cu. ft., are on the Allegheny Basin, and four, with a total capacity of 17 302 900 000 cu. ft., are on the Monongahela Basin, the total capacity on the two basins being 59 481 400 000 cu. ft. The total cost of the seventeen reservoirs would be $21 672 100, or an average cost of $364 per 1 000 000 cu. ft. of storage capacity.

The locations of these seventeen selected projects, together with the drainage area controlled by each, are shown on Fig. 2.

It was found that, if these seventeen projects had been constructed, the average reduced peak for the eleven floods would have been 18.9 ft.; and that in only the two highest floods, March 15th, 1907 (35.5 ft.), and March 1st, 1902 (32.4 ft.), would the reduction have been less than to the danger line of 22 ft. In the 1902 flood the reduced peak would have been only 0.3 ft. above the danger line, and would have been above that line for such a short time as to be negligible. The reduced peak of the 1907 flood, however, would still have been 6.8 ft. above the danger line, and a certain amount of overflow and damage might still result from occasional great floods, if the flood relief work should be confined to the construction of the seventeen selected reservoirs.

Combination of Reservoirs and Low Wall.—The reduction of the maximum flood to this height, or even to 30.0 ft., however, would greatly reduce the amount of wall necessary to prevent overflow. Investigation showed that the cost of the low wall which could be used in combination with the seventeen selected reservoirs would be so small that it would be far cheaper to construct it than to build the large amount of additional reservoir storage which would be necessary to reduce the occasional great floods below the danger line.

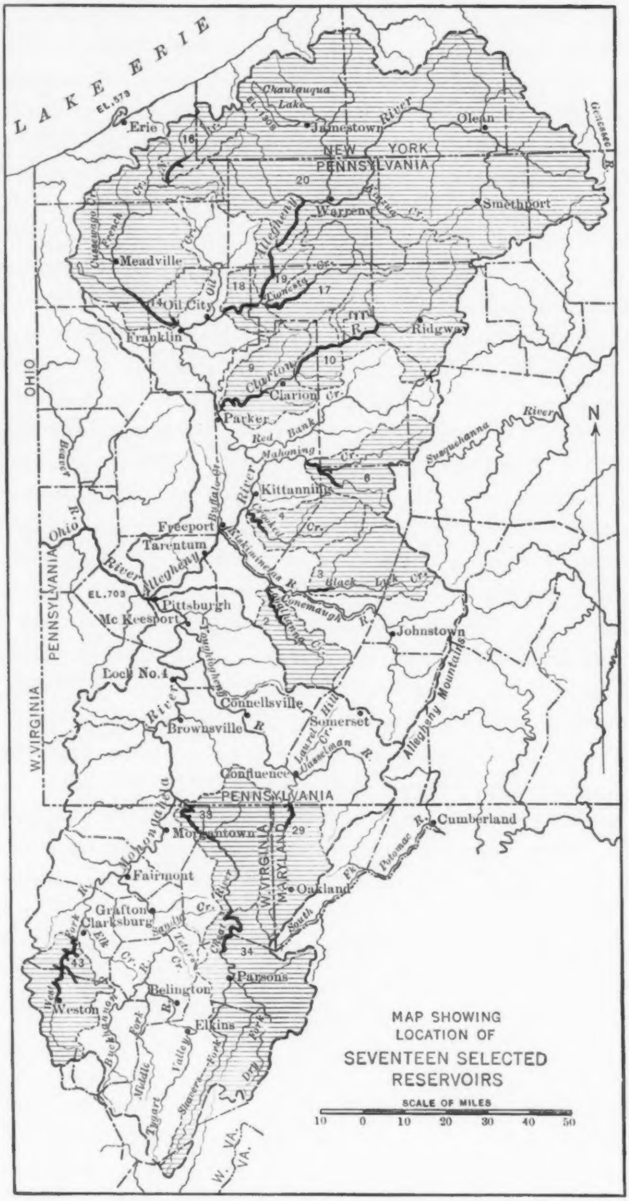

LAKE ERIE
EL.573
Erie
Chautauqua Lake
EL.1308
Jamestown
River
Olean
NEW YORK
PENNSYLVANIA
Warren
Smethport
Meadville
French Cr.
Oil City
Franklin
Tionesta
Allegheny
Ridgway
Clarion
Clarion
Parker
Red Bank
Mahoning Cr.
Kittanning
Crooked Cr.
Freeport
Tarentum
Kiskiminetas R.
Black Lick Cr.
Conemaugh R.
Johnstown
Pittsburgh
EL.703
Mc Keesport
Ohio R.
Beaver
OHIO
PENNSYLVANIA
W.VIRGINIA
Lock No.4
Connellsville
Somerset
Brownsville
Laurel Hill Cr.
Casselman R.
Allegheny Mountains
Confluence
Susquehanna River
Cumberland
Potomac R.
South Fk.
Morgantown
Monongahela
Fairmont
Oakland
W.VIRGINIA
MARYLAND
Grafton
Clarksburg
Cheat River
Parsons
Belington
Elkins
Weston
West Fork
Buckhannon
Tygart
Valley
Shavers Fork
Dry Fork
W. VA.
VA.
N
MAP SHOWING LOCATION OF SEVENTEEN SELECTED RESERVOIRS
SCALE OF MILES
10 0 10 20 30 40 50

FIG. 2.

These seventeen selected reservoirs would reduce the flood heights so much that no wall would be required on the Monongahela or Ohio Rivers. It would be necessary to construct a wall only along a few low-lying portions of the Allegheny. This low wall, in combination with the reduction in flood heights which would be obtained by reservoir storage, would prevent overflow by a 40-ft. flood at Pittsburgh. It would cost about $670 000. The total cost, therefore, of the wall and reservoirs would be $22 342 100.

If certain revisions in the river channels were made, the total value of the land reclaimed along the river front would amount to about $2 307 000. The total net cost of the low wall and the seventeen selected reservoirs combined, therefore, would be $20 035 100.

Reduction of the Flood of March 22d, 1912.—The flood of March 22d, 1912, is the first of any importance that has occurred since gauging stations have been in operation on all the tributaries controlled by the seventeen selected reservoirs referred to. The accompanying graphical estimate (Fig. 3) of the probable reduction in the Pittsburgh peak which could have been obtained had the system of reservoirs recommended by the Flood Commission been in operation is, therefore, of peculiar interest, as confirming the claims made by the Commission and showing the conservative character of the assumptions made necessary by incomplete data in certain portions of the above described flood-reduction studies.

The peak reduction diagram, Fig. 3, is similar to the diagrams used in the studies made by the Flood Commission. The discharges at Pittsburgh were obtained by gauge heights furnished by the U. S. Weather Bureau, and an approximate discharge curve for the Ohio River at Pittsburgh was based on measurements by the U. S. Engineers. The flood-volume curves of the tributaries controlled by the seventeen selected projects are, with two exceptions, based on actual stream-flow measurements. The curve representing the flood discharge of the Allegheny River at the site of the lower of the three Allegheny projects, a short distance above Oil City, is based on discharge measurements at Red House, N. Y.; the Loyalhanna Creek curve is based on the records of Black Lick Creek and a study of rainfall data, the Loyalhanna gauge having unfortunately been out of commission during this flood.

These tributary flood-volume curves, representing the flood-water which would have been held back by the proposed reservoirs, are plotted

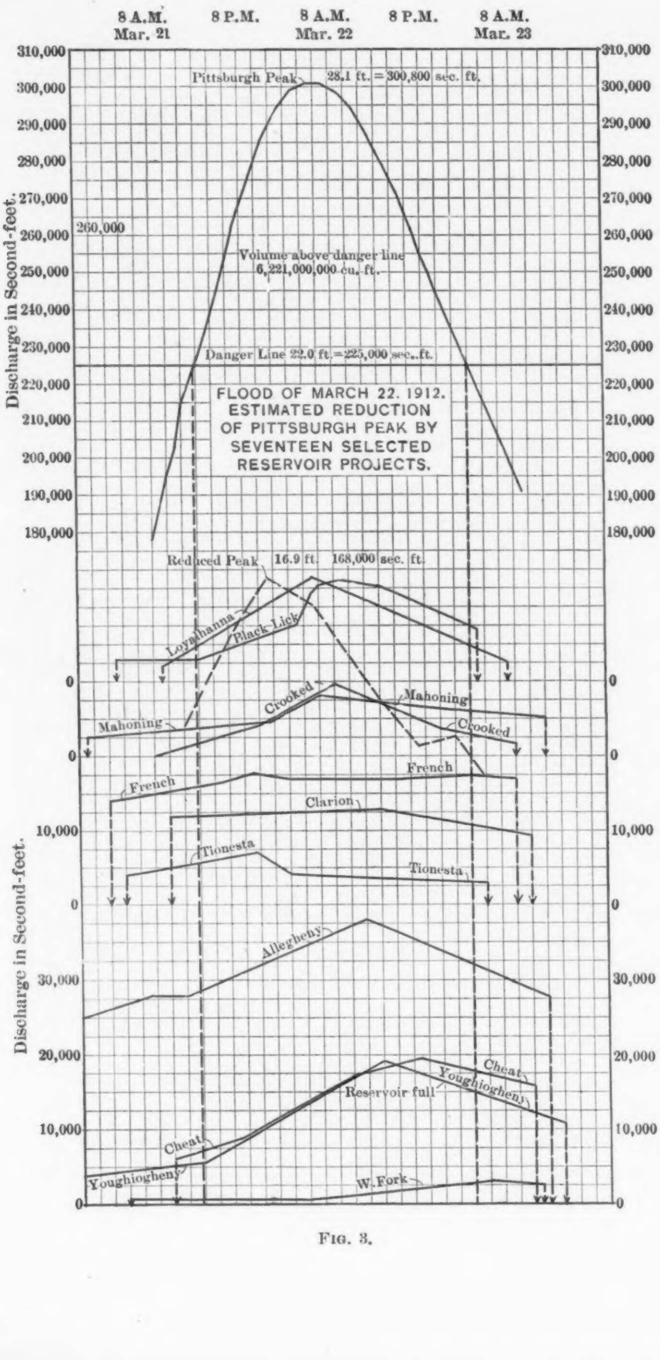

8 A.M. Mar. 21
8 P.M.
8 A.M. Mar. 22
8 P.M.
8 A.M. Mar. 23
Discharge in Second-feet.
Pittsburgh Peak
28.1 ft. = 300,800 sec. ft.
260,000
Volume above danger line 6,221,000,000 cu. ft.
Danger Line 22.0 ft. = 225,000 sec. ft.
FLOOD OF MARCH 22. 1912. ESTIMATED REDUCTION OF PITTSBURGH PEAK BY SEVENTEEN SELECTED RESERVOIR PROJECTS.
Reduced Peak 16.9 ft. 168,000 sec. ft.
Loyalhanna
Black Lick
Crooked
Mahoning
French
Clarion
Tionesta
Allegheny
Cheat
Youghiogheny
Reservoir full
W. Fork

FIG. 3.

under the Pittsburgh peak in the positions in which it is estimated they actually arrived at Pittsburgh, based on the calculated time of travel of floods from the respective dam sites to the latter place. On account of their smaller size, they are plotted to a vertical scale twice as great as that used for the Pittsburgh peak. By subtracting the sum of their ordinates at a series of points, that is, one-half the graphical sum, on account of the difference in scales, from the corresponding ordinates of the Pittsburgh peak, a reduced peak has been obtained, the highest point of which represents a discharge of only 168 000 sec-ft., corresponding to a gauge height of 16.9 ft. This represents a reduction in discharge of 132 800 sec-ft., or 43%; and in gauge height, of 11.2 ft., or to 5.1 ft. below the danger line.

Table 3 contains various data with regard to the seventeen reservoirs. It also includes the results of an analysis of the peak reduction diagram, and brings out the relative effectiveness of the respective projects in reducing this particular flood.

The term, "effective capacity," has been used to designate that part of each tributary flood-volume curve falling within the effective zone, or the time when the Pittsburgh crest was above the danger line, or 22-ft. stage. In other words, it represents the portion of the capacity used in storing damaging flood-water. The reduction in gauge height to be credited to a given project has been obtained by multiplying the total reduction, 11.2 ft., by the percentage of total effective capacity credited to each project.

It will be noted that, with the exception of the Youghiogheny project, all the reservoirs are of ample capacity to have controlled the damaging flood-water delivered from their respective catchment areas. The Youghiogheny reservoir would have filled up a few hours before it ceased to deliver damaging flood-water. The position of this filling up has been indicated on the peak reduction diagram, Fig. 3; and, beyond this point, the ordinates of the Youghiogheny flood volume curve have not been used in reducing the Pittsburgh peak.

About 293 000 000 cu. ft. of additional storage capacity would have been needed to store all the damaging flood-water delivered by the Youghiogheny above the reservoir. The surveys showed that, by increasing the height of the dam to 96 ft., about 984 000 000 cu. ft. of additional storage could be obtained at this site, and the capacity of this reservoir should be increased by part of this amount, at least, in order to insure complete control of all floods from its catchment area.

TABLE 3.—REDUCTION OF FLOOD OF MARCH 22D, 1912, AT PITTSBURGH BY STORAGE RESERVOIRS.

Number indicating reservoir on Fig. 3.	Project.	Height of dam, in feet.	Capacity, in millions of cubic feet.	Cost.	Drainage Area Controlled.		Percentage of total cost.	Effective Capacity.			Reduction in gauge height.	Cost per Million Cubic Feet of Capacity.		Ranked According to:		
					Square miles.	Percentage of total above Pittsburgh.		Millions of cubic feet.	Percentage of total capacity of project.	Percentage of total effective capacity of all projects.		Of total capacity.	Of effective capacity.	Cost per million cubic feet of total capacity (lowest = No. 1).	Cost per million cubic feet of effective capacity (lowest = No. 1).	Effectiveness. (Greatest = No 1.)
2	Loyalhanna........	122	4 112.5	$1 222 000	277	2.4	5.6	1 242.0	30.2	7.52	0.84	$297	$984	7	3	4
3	Black Lick..........	63	1 454.7	720 500	414	3.6	3.3	1 188.0	81.7	7.17	0.80	495	607	13	1	6
4	Crooked............	94	3 255.7	893 700	287	2.4	4.1	727.2	22.3	4.40	0.49	274	1 229	5	6	11
6	Mahoning...........	143	2 367.8	1 089 400	335	2.9	5.0	828.0	35.0	5.00	0.56	460	1 315	12	8	10
9	Clarion No. 1........	142	5 067.1	1 309 900	1 212	10.5	6.1	691.2	13.6	4.18	0.47	258	1 895	4	14	12
10	Clarion No. 3........	128	4 886.6	1 028 100	......	....	4.8	666.0	13.6	4.03	0.45	210	1 544	1	11	13
11	Clarion No. 4........	70	1 537.9	451 800	......	....	2.1	208.8	13.6	1.26	0.14	294	2 164	6	15	16
14	French............	75	3 323.1	2 388 100	1 009*	8.6	11.0	1 836.0	55.2	11.10	1.24	719	1 301	17	7	2
16	No. Br. French......	67	2 125.7	722 800	217	1.9	3.3	396.0	18.6	2.40	0.27	340	1 825	9	13	15
17	Tionesta	103	3 629.6	1 362 600	477	4.1	6.3	601.2	16.6	3.64	0.41	375	2 266	10	16	14
18	Allegheny No. 1.....	63	2 876.3	1 219 900	3 795†	32.8	5.7	1 231.2	43.0	7.45	0.83	424	991	11	4	5
19	Allegheny No. 2.....	66	4 877.9	3 057 000	......	....	14.1	2 095.2	43.0	12.67	1.42	627	1 459	15	9	1
20	Allegheny No. 3.....	54	2 663.6	1 386 000	......	....	6.4	1 144.8	43.0	6.92	0.78	520	1 211	14	5	7
	Total Allegheny....	.....	42 178.5	$16 851 800	8 023	69.2	77.8	12 855.6	30.4	77.74	8.70	$400	$1 311	..	..	..
29	Youghiogheny......	76	1 547.1	$999 300	394	5.4	4.6	1 547.1	100.0	9.36	1.05	$646	$646	16	2	3
33	Cheat No. 1.........	113	5 737.4	1 287 900	1 399	19.1	5.9	849.6	14.8	5.14	0.58	224	1 516	2	10	9
34	Cheat No. 2.........	136	7 294.1	1 721 900	......	....	8.0	1 080.0	14.8	6.52	0.73	236	1 594	3	12	8
43	W. Fork............	66	2 724.3	811 200	366	5.0	3.7	205.2	7.5	1.24	0.14	298	3 953	8	17	17
	Total Monongahela.	.....	17 302.9	$4 820 300	2 159	29.5	22.2	3 681.9	21.2	22.26	2.50	$279	$1 309	..	..	..
	Grand total......	.....	59 481.4	$21 672 100	10 182	53.8	100.0	16 537.5	27.8	100.00	11.20	$364	$1 310	..	..	..

NOTE.—Total drainage area of Allegheny River is 11 580 sq. miles; of Monongahela River is 7 340 sq. miles.

* Excluding area controlled by No. Br. French project. † Excluding area controlled by Tionesta project.

It should be noted, however, that before the discharge of about 17 000 sec-ft., which the Youghiogheny was delivering at the time the reservoir would have filled up, could have been reached in the stream below the dam, enough water would have been stored in the reservoir to give a head of about 2.5 ft. on the spillway. With a surface area of 940 acres, this would have given an additional storage of about 102 000 000 cu. ft.

The costs per 1 000 000 cu. ft. of total and of effective capacity are also shown in Table 3. In the last three columns, the projects are ranked according to these costs, the lowest being No. 1; and, according to effectiveness, the project with the greatest reduction in gauge height to its credit is ranked as No. 1.

Evidently, it does not follow that, because a reservoir has a low cost per 1 000 000 cu. ft. of storage, it is the cheapest to build for flood-control purposes. The water which it controls may be that of a tributary which rarely, if ever, delivers that flood-water at Pittsburgh during the critical period of damaging flood height. Whereas, another reservoir, of high cost per unit of storage, may control a stream which is invariably an offender whenever its drainage area receives flood rainfall. A number of favorable reservoir sites having been selected and surveyed, and estimates of the capacities and costs of the respective projects having been determined, if only a certain number of them are to be built, the ultimate criterion for their selection for flood-control purposes is not the lowest cost, nor even the greatest effectiveness; it is both. In other words, the final selection should be based on the lowest cost per unit of effectiveness.

Thus, in Table 3, it will be noted that, in this particular flood, the Clarion No. 3 project ranked 13 in effectiveness and 11 in cost per unit of effectiveness, although it ranked 1 in cost per unit of total capacity. The Youghiogheny project, on the other hand, although it ranked 16 in cost per unit of total capacity, being exceeded only by the French Creek project, ranked 3 in effectiveness and 2 in cost per unit of effectiveness.

It is obvious, of course, that such a selection of reservoirs cannot be made from a study of one flood, for no two floods are alike in their origin. The rainfall varies widely in amount, in distribution, and in relative time of arrival. Moreover, even with the same rainfall, the run-off from a given part of a drainage area, being affected by

many conditions, may vary widely. It is necessary, therefore, that the volume and distribution of storage shall be adequate to control all possible conditions and combinations of run-off.

It was in the foregoing manner, after making a similar analysis of the action of the forty-three reservoirs in the eleven principal floods from 1898 to 1908, that the seventeen reservoir projects recommended by the Flood Commission of Pittsburgh were selected. As previously stated, it is estimated by the Commission's engineers that, with these seventeen reservoirs, all floods at Pittsburgh, including a possible future 40-ft. flood, would be reduced to a point where overflow and flood damage could be prevented by a low wall along a few low-lying portions of the Allegheny River; and that the total net cost of these reservoirs and the wall, taking into account the value of the land reclaimed, would be only about $20 000 000, or less than twice the direct losses from flood damage sustained during the past ten years at Pittsburgh alone. Moreover, the flood relief would not be confined to Pittsburgh, but would be extended along the rivers and their principal tributaries above that city, and down the Ohio River for many miles below it. For example, at Wheeling, W. Va., 90 miles below Pittsburgh, the greatest recorded flood, that of 1884, 53.1 ft., could have been reduced by 13 ft.; and the next greatest, that of March, 1907, 50.1 ft., by 14.5 ft.

Proceeding farther down the Ohio, it is evident that, as other important tributaries enter, this storage would become a decreasing factor in reducing the Ohio floods. In so far as such floods originate above Pittsburgh, the storage would be effective, but complete control of floods at these lower points would necessitate the construction of additional storage on the tributaries entering the Ohio between Pittsburgh and the point where flood relief is desired. A similar solution of the problem on the other tributaries of the Ohio would extend the flood relief throughout its entire valley.

Above Pittsburgh, the Allegheny River, from its mouth to Oil City —a distance of about 134 miles—would be relieved from frequent disastrous floods. Flood levels on the Kiskiminetas River, for a distance of 41 miles above its mouth, would also be considerably lowered. On the Monongahela River, floods would be greatly reduced from Fairmont, W. Va., to Pittsburgh, a distance of 127 miles. On the West Fork River, they would be controlled for a distance of 38 miles above the mouth, and, on the Youghiogheny River, they would be lowered considerably for a distance of 83 miles above its mouth.

The system of storage reservoirs could be operated primarily for flood prevention during the flood season, and for increasing the low-water flow during the low-water season. A study of the flood records shows that 80% of the floods has occurred in the months from November to March, inclusive; and that the 20%, occurring in the period from April to October, inclusive, has been of relatively less severity, and could have been prevented from doing damage by the low wall which has been proposed in combination with the reservoirs.

With the amount of flood-water which, by this method, could be safely retained in these reservoirs until needed in dry weather, combined with the surplus spring and early summer flow that could be gathered, a notable improvement in the low-water flow of the main rivers and their tributaries could be effected. The low-water flow of the Allegheny at Pittsburgh could be increased to about 3.5 times its minimum throughout the entire dry-weather season; that of the Monongahela to about 6 times its minimum; that of the Kiskiminetas River to about 6 times its minimum; and that of the Youghiogheny River to about 10 times its minimum. These increases of low-water flow are those which would obtain with only the seventeen selected projects constructed, and would, of course, extend down the Ohio. At Wheeling, the low-water discharge could be maintained at 3 times the present minimum, corresponding to an increase in stage of 2.3 ft.

The extent of these improvements in low-water flow will be appreciated when it is realized that 267 miles of main rivers and 386 miles of tributaries, or a total of 653 miles of stream channels above Pittsburgh, would have their low-water discharge considerably increased and made uniform during dry weather. If all the forty-three projects referred to were constructed, there would be, of course, a considerably greater increase of low-water flow, which would extend over 393 miles of main rivers and 570 miles of tributaries, or a total of 963 miles of stream channels above Pittsburgh.

The benefits to navigation, sanitation, water supply, and water-power, which would result from such an improvement in stream regimen would naturally be very considerable. They form a strong additional argument in favor of the construction of storage reservoirs for purposes of river regulation and flood control, and broaden the problem to one demanding State and National consideration.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

ADDRESS AT THE 44TH ANNUAL CONVENTION, SEATTLE, WASHINGTON, JUNE 25TH, 1912.

By John A. Ockerson, President, Am. Soc. C. E.

Sixteen years ago this month the Annual Convention of the American Society of Civil Engineers was held for the first time on the Pacific Coast, and our second visit has perhaps been unduly delayed, although we have for years looked with longing eyes toward the time when a meeting could again be held on these hospitable shores.

There is a tradition among the good people of this Western World that those who make a third visit become so enamored of the advantages, the attractions of the climate, the material resources, and the people, that they refuse thereafter to leave it, but in turn become enthusiastic advocates of life in the Far West.

Perhaps it is just as well, therefore, that the intervals between meetings on the Pacific slope are somewhat long.

It is, however, well worth a long journey to any of our Annual Conventions and our Annual Meetings, to exchange greetings with old friends and acquaintances, and to find new ones. This applies with equal force to the whole country, be the distance from the Society House to the place of gathering great or small.

My membership in this Society dates back to the Twelfth Annual Convention, which was held in St. Louis in 1880.

Since that time thirty-one presidents of the Society have delivered their annual addresses, covering the whole range of duties and relations of members to the Society, the purposes of the Society, its

influence in elevating the Profession in public esteem as the peer of the so-called learned professions, and its important function in establishing the unity of the Profession of Civil Engineering, although many organized engineering specialties have developed other societies with large and active membership.

The important place which the engineer has occupied and will continue to occupy in the development of our country, by directing the "great sources of Power in Nature for the use and convenience of Man," has been repeatedly discussed in convincing manner.

Codes of ethics and laws to govern the practice of Civil Engineering have been ably treated on several occasions.

Some of the presidential addresses were prophetic in their estimate of what the future would bring forth; and the progress in the field of engineering cannot be better illustrated than by citing a few of the predictions that have been more than realized, even during the comparatively brief period of my membership in this Society.

Some of you will doubtless remember that the late Octave Chanute, who was Vice-President of the Society in 1880 and later became President, was the American pioneer in efforts to solve the problem of aerial flight by man.

In the Presidential address,* prepared by him in that year, he stated:

> "I suppose you will smile, when I say that the atmosphere yet remains to be conquered; but wildly improbable as my remarks may now seem, there may be engineers in this room, who will yet see men, safely sailing through the air."

He lived to see the fulfillment of this prophecy himself, so far as relates to the possibility of navigating the air; but there is evidently still much to be desired as to the safety of those who travel in aeroplanes, as well as "they that go down to the sea in ships."

Another President of the Society, a year later, quoted the prediction of Sir William Thomson, that the time would come when the Falls of Niagara would be largely utilized for light and power over a large area of North America; that a half-inch copper wire would transmit 21 000 horse-power to Montreal, Boston, New York, or Philadelphia.

The remarkable developments in the application of electricity for

* *Transactions*, Am. Soc. C. E., Vol. IX, p. 255.

the propulsion of railway trains and street cars, for lighting cities, for electric furnaces, and for motive power of various kinds, all demonstrate that the prediction made has been greatly exceeded by actual progress in many lines.

Water-power plants for the development of electric energy of far greater capacity than were even dreamed of by Sir William Thomson are now in operation or in process of construction in many parts of our country.

One of the greatest of these is the power plant now under construction at Keokuk, where a concrete dam across the Mississippi River will result in developing 200 000 horse-power by converting the energy of the flowing stream into electric currents which may be transmitted to distant points for use.

In another address of this period, the development of elevators for successful use in high buildings was heralded as a remarkable example of progress in engineering, while they have long since come into such general use as to be no longer a luxury, but a necessity, required in every building of even moderate height.

In the vicinity of New York, ferry-boats have been largely replaced by great bridges and tunnels, which are to-day striking examples of the skill of the engineer. In the same locality enormous sums are being expended on terminals to facilitate the movement and interchange of traffic. Neither is the health of the people of our greatest city lost sight of, as is shown by the gigantic work now in progress which will provide an ample supply of water from sources free from pollution.

A similar work of large proportions is nearing completion for the City of Los Angeles, the details of which will be presented to the Society at this Convention.

Work on the Panama Canal, the greatest project of this century, is progressing rapidly. The problem of an Isthmian Canal, which received the serious consideration of several nations for many years, with a great diversity of opinion as to its proper location, its dimensions, its character—as to whether it should be at sea level or be constructed with locks—is soon to be realized as a successful work of American engineers, which will be of vast benefit to the world at large, even if not profitable to us as a business venture. Less than twenty years ago, the President,* in his annual address, stated that, in

* William P. Craighill, President, Am. Soc. C. E., *Transactions*, Vol. XXXI, p. 568.

his opinion, the day was not far distant when the United States would construct, own, and hold a ship canal across the Isthmus, and that then, perhaps, our domain would be extended to make that canal part of our southern boundary.

The great irrigation works which are converting the desert into fertile fields are essentially projects of to-day that engage the highest skill and energy of the engineer.

A remarkable feat in railway building is to be found in the road over which we have been carried in comfort and safety to this beautiful city.

Rivers and deep gorges have been spanned by substantial bridges, mountain ranges have been pierced by tunnels, and an excellent roadbed has been constructed.

That portion of the line extending from the Missouri River to Seattle, about 1 376 miles in length, was built within a period of three years.

Much credit is due the engineers connected therewith for the satisfactory manner in which the many perplexing problems have been solved.

It is not my purpose, however, to attempt the enumeration of all the great engineering works of the present day, even in our own country. It is gratifying to note that the members of this Society are conspicuous in them all.

Originally, the Constitution required the President, in his address, to give a review of the progress of engineering during the preceding years, but the development was so rapid and the field became so wide that the rule was amended so as to require merely an address.

From what has preceded, I am sure you will sympathize with me in my vain efforts to find a subject, relating to our well-being as a Profession and as a Society, that has not already been ably and exhaustively treated by my predecessors, and you will appreciate the necessity and propriety of limiting my remarks largely to the Society in its present relation to all the members, and the desirability of cultivating a closer personal interest in its management and a pride in its membership.

At the beginning of my membership, the Society numbered 600 members, and only 30 were added during that year, while at the present time the total membership is 6 512, with a gain of 529 during 1911.

In this gain the three Pacific Coast States are conspicuous with a present membership of 647, or nearly one-tenth of the total membership.

California, with its 407 members, ranks third in point of numbers, being only exceeded by New York and Pennsylvania, and during the year 1911 it had the largest increase in membership of any State except New York.

The State of Washington has 136 members. The States west of the Mississippi have 1 829 members, while the State of New York alone has 1 755 members, and there are 1 388 resident members living within fifty miles of the post office of New York City.

The preponderance of numbers in the Far East doubtless accounts for the preponderance of influence and interest in the affairs of the Society which is sometimes credited to them.

Members of our Society are also to be found in 36 foreign countries. So it will be seen that it has grown in influence as well as numbers, and still continues to grow at a rapid rate.

It has been said that our Society is too conservative, and as a result new societies have been formed, which could have been satisfactorily provided for within our own organization.

There seems also to be a disposition to avoid participation in the discussion of public questions, even when closely related to the work of the Profession. When Congressional Committees call on the Society for advice with regard to pending legislation, involving questions relating to engineering, it would seem to be a proper function of the Society to render such aid as may be practicable.

In fact, it might be well, under proper conditions, to go even farther, and use the influence the Society may have to mould public opinion along lines free from local or political bias, when our public works are the subject of discussion.

These and other problems bearing on the aims of the Society, have often been discussed, and will continue to receive consideration, with a view to increasing its usefulness.

The influence of our Society will always depend on the character of its membership, and to this end every one connected with the Society should, as far as may be practicable, take a personal interest in the admission of its members. The Board of Direction must rely

on the members for the information needed to judge of the professional and personal fitness of an applicant for admission.

If this duty, common to all members, is conscientiously performed, the simple fact of being a member of the Society will be of far greater value than any license that could possibly be given by legal enactment.

The only good purpose to be served by a system of license, to authorize an engineer to practice his profession, would be to protect the employing public from incompetent men.

Membership in the Society, if carefully guarded, would soon be accepted as a far better guarantee of fitness than any license system that could be devised, and the expense, to both the public and the engineer under such system, which would be a considerable sum, would be wholly unnecessary, as Membership would at once be a certificate of qualifications to design and direct engineering work successfully. The Society itself should be the first to condemn an unworthy member, but should be equally prompt in defending a member who is unjustly accused of wrong doing.

As a matter of fact, the Society is already regarded by the public so highly that its members are looked on with special favor by the Courts when expert testimony is required, by the Government when seeking for capable men for service on public works, and by municipalities where men of integrity and ability are looked for to fill positions of trust relating to the engineering side of city government.

In a speech at the Society House recently a leading politician of New York announced that they have come to realize that the interests of the city are best served by appointing engineers to fill the important offices which have charge of the physical welfare of the city in general.

This is true of many of our cities, and the Profession is steadily growing in public favor; loyalty to the Society on the part of all its members wherever they may be located will greatly stimulate this growth.

No matter how remote you may be from the House of the Society, do not for a moment harbor the feeling that nothing is gained by membership therein, except the copies of the *Transactions* and the privilege of hanging up a certificate of membership and wearing the Society badge.

The Society belongs to you, and a personal interest in all its affairs will make a better Society as well as a better member. The Board of Direction, which represents you, will welcome such interest, to the end that the usefulness of the Society may be broadened.

It is said by some that the Society is not truly National because it has its house in New York where are held the Annual Meetings and the meetings where papers are read.

That the proper place for the home of the Society is in the Metropolis of our country can hardly be questioned. In no other city could there be found such a large resident membership, which equals one-fifth of the total membership. The excess dues charged to such members cover nearly one-third of the entire cost of maintaining the Society. So, from a purely business point of view, the situation must be regarded as satisfactory.

If ways and means can be devised to better the practice as to meetings, it lies in the power of the members to have it done.

The speaker has given some study to this question, and finds a possible remedy in more frequent conventions or meetings in different localities readily accessible to a considerable number of members.

The distances are so great that a convention is necessarily made up almost wholly of members comparatively near at hand. This is more particularly true since the system of free transportation, formerly generously accorded to us, has been abolished. Although the Society has greatly increased in numbers, the attendance at conventions has decreased, due chiefly to the cost of transportation, which was formerly free.

Take, for example, the convention this year on the Pacific Coast. Comparatively few members can devote the time and expense involved in a trip from near the Atlantic seaboard, and if another convention were to be held in the East a month later, there would doubtless be a larger attendance than we have here to-day.

More local associations, for reading and discussing papers, would also add materially to the interest of the members in the Society; but, even with these, the acquaintance and affiliation of members from different localities would not be realized, and this is one of the chief advantages of general gatherings, such as our Conventions and Annual Meetings, that can hardly be reached in any other way.

The proper solution of this question is worthy of your best

thoughts, and when you have reached satisfactory conclusions, give the Board of Direction the benefit thereof.

The selection of a suitable place for the Annual Convention has often been quite embarrassing, as many localities are generally named in response to requests for suggestions from members as to the most satisfactory place for such meeting.

Many places are usually suggested, and it is rarely that any one place receives such a majority of votes as to indicate a decided preference, and the matter, after all, must be decided by the Board of Direction.

It not infrequently happens that the members in a given district feel slighted because their invitation is not accepted, and feel that favoritism is shown in making the selection. Whether there is just ground for such belief or not, a remedy lies in the hands of the members.

Suppose we let the general locality of the Annual Convention be fixed in the several districts in certain proper numerical order, and let the members of each district determine the specific locality therein where the meeting shall be held. The location will then be automatic to a considerable extent, and no one need feel slighted.

Whenever special reasons of sufficient moment should make it desirable to waive the regular order of progression, it seems hardly probable that serious objection thereto would be urged. A friendly rivalry between the different districts might be developed by such method of selection, which might prove to be highly beneficial.

These suggestions are made in the hope of stimulating the personal interest of all members in the management of the affairs of the Society, not only as a privilege, but as a duty.

It is related of Telford, the greatest engineer of his time, that he offered no encouragement to young men who aspired to enter the Profession of Civil Engineering. On the contrary, he told them there was nothing left to do in Great Britain, as he had already built all the canals, roads, and harbors that the country required.

He died seventy-eight years ago, and the intervening years have witnessed the planning and construction of far more and greater works than were even dreamed of in his day and generation, and the end is not yet.

In our own country, the developments in many lines have been

so rapid, and projects of great magnitude have been conceived, planned, and completed so rapidly, that there would seem to be an end thereto; but, day by day come new projects. Existing works have become obsolete, or the capacity has become too limited to satisfy the needs of the rapid increase in population, hence we must tear down the old and build larger and better. These changes are so rapid that it requires looking well into the future to determine, as best we can, how to build the works of to-day so as to meet the requirements of to-morrow.

The locks at Sault Ste. Marie which serve the traffic of the Great Lakes have been rebuilt and enlarged four times to meet the increased demands of the great volume of traffic which passes through them.

Had the Panama Canal been built on the lines laid down by De Lesseps, it would have been obsolete before it could have been completed, and reconstruction on a larger scale would have been imperative.

Much of our own work in the past has been done with little reference to permanency, but rather to meet a present emergency, and such work must all be reconstructed or better work substituted. The sharp curves and steep grades of our railways must be eliminated; the bridges and tracks that were ample for our traffic of light locomotives and 20-ton cars of past years, must give way to more substantial structures that will carry the giant engines of to-day hauling long trains with car loads of 50 tons.

The channels and harbors with 20-ft. depth, which answered the purpose a few years ago, must be developed to depths of from 35 to 40 ft.

The cities have grown from a few thousand souls to hundreds of thousands and even millions, and the water supply, the sewers, the streets, and all that is required for the health and comfort of the people must keep pace therewith. New cities are also springing up, and these must be equipped with modern appliances.

Our Government has yet to adopt a definite policy as to the improvement and flood control of our numerous waterways. While considerable work has been done on the larger streams, it is trifling in amount as compared with what must yet be done in the development of streams for navigation, in the conservation and use of the

waters for power, in the control of floods, and in the drainage of wet lands.

Then, too, a large portion of our territory is yet undeveloped, so it is apparent that the demands for engineers will continue to increase, and the rewards for their labors will continue to grow.

It seems evident that there is still far more work for the Civil Engineer to do than has yet been accomplished, and a bright and busy future is assured for many years to come.

In this development our Society has important obligations to fulfill to the Profession and to the public. It should fix and require such moral and professional standards that membership will at once be recognized as a guarantee of integrity, honor, and ability, so that its members may merit and receive the fullest respect, esteem, and confidence of their fellow-men.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852.

PAPERS AND DISCUSSIONS

THE JUST VALUE OF MONOPOLIES, AND THE REGULATION OF THE PRICES OF THEIR PRODUCTS.

Discussion.*

By Joseph Mayer, M. Am. Soc. C. E.†

Joseph Mayer, M. Am. Soc. C. E. (by letter).—The criticisms submitted make it plain that some of the writer's ideas have not been presented with sufficient clearness to convince the reader of their truth. The writer's fault lies partly in inadequate fullness of definitions, and this has caused a consequent confusion of the critics. Mr. Mayer.

The fundamental ideas discussed in the paper are value, justice, monopoly, and regulation of prices. To avoid confusion, it is necessary to use each of these terms with a definite meaning.

Value.—Man wills to live. Things which further life are useful. Useful things which are difficult to obtain are valuable. Usefulness and difficulty to obtain are the two necessary elements of value, but, as both are essential, neither can be used as a measure of value. The value of a thing is measured by the quantity of other valuable things which can be obtained for it in free exchange. Values are extremely variable until there is a market where many purchasers and sellers meet. Things are exchanged for money; the amount of money a thing sells for is its price. When all prices at a given time and place are known, all the values are known. As long as the value of money remains the same, the prices are correct measures of values. Things have three prices, the bid, the asked, and the price of actual transactions. The last lies somewhere between the former two. The man who is obliged to sell will accept the bid price, and the man who is obliged to buy will pay the asked price. These prices differ widely

* Continued from May, 1912, *Proceedings*.

† Author's closure.

Mr. Mayer. where there are but few sellers and purchasers of a commodity. Speculators, who watch the markets, buy from urgent sellers and sell to urgent buyers, reduce these differences, and are thereby useful. The usefulness of a thing gives the upper limit of the price bid. For things produced in large quantities, by many independent producers, cost of production by common methods plus ordinary profits gives the average price asked. The usefulness of a means of production consists in furnishing a net income to the owner in the present and during a limited future. The price paid for a means of production approximates the estimated present value of the net income it will give to the owner.

The net earnings of regulated monopolies depend on the prices permitted for their products and their cost of production. The values of these monopolies depend on these net earnings and the rate of interest, which latter must be known in order to determine the present value of future net earnings. The cost of production of the plant of a monopoly gives no indication of its value unless it influences either the present or the future regulation of the prices of its products. The value of a plant for making competitive products depends also on the present value of its future net earnings. This is largely influenced by the cost of production, with ordinary ability, not of a similar plant, but of one making similar products and securing net earnings of equal present value. The value of such a plant is governed mainly by its capacity to secure a low cost of production for valuable products. It depends at least as much on its location and the arrangement of its parts as on their cost, as much on the design as on the quantity and quality of its material parts. The value of the whole, therefore, is something entirely different from that of the parts or their cost of production. A complicated and old plant may be misplaced for present conditions, and will often contain many features ill adapted to the present most economic methods of production, and other features, the result of eliminations of unsuitable uneconomic arrangements, which may make it, if it has had able managers in the past, a much more economic plant than a new one of equal cost designed by a manager of but ordinary ability. A plant is largely a mental product, and its value depends on the quality of the designers' minds. This quality cannot be measured by a physical valuation based on cost of production or reproduction; it is only measurable by the net earnings secured. This is felt by all who attempt to ascertain the value of an industrial enterprise by a physical valuation. The absurd results obtained by such a valuation make it necessary to correct them by introducing other elements, such as depreciation, going value, intangible value, franchise value, etc. All these can only be estimated by comparing the present value of all its estimated future net earnings with that found by a mere

physical valuation. The difference is the supplementary value of the many names, which, added to the physical value, gives again the real value used in finding the supplementary value. The physical valuation, therefore, is entirely superfluous. Mr. Mayer.

Depreciation is sometimes defined independently as a physical quality, without considering the net earnings secured by the plant or by that part of it. The average useful life of a machine or building is estimated, and the present value is ascertained by subtracting from the value when new a rate of annual loss which will reduce it to nothing when the machine is worn out. A machine usually becomes worthless, not by what happens in it, but by what happens outside of it; it becomes antiquated because of new inventions and methods of production at different rates; and it is often evident that a machine or building has become antiquated and has lost value much more rapidly or slowly than previously estimated. It would be misleading to stick to a valuation based on uniform rules after new facts which prove them incorrect are known.

The importance of the new facts can only be estimated by the change of net earnings which they cause. The rule given for determining depreciation, therefore, is either untrue or dependent on future net earnings. The statement that the value of an industrial plant or a business is equal to its cost of production less depreciation, or to the cost of reproduction of a new plant of the same kind less the physical depreciation of the old one, therefore, is untrue unless additional elements of an immaterial nature are introduced, which practically change it to the rule that its value is equal to the present value of its future net earnings. For enterprises, the securities of which are largely dealt in on the exchanges, the quotations give the best available estimate of this value. They are the value, because they give the price for which known fractions of the total value of the business are sold.

To change by regulation the future prices of the products of an industrial enterprise, and, consequently, its net earnings, is to change its value, and an attempt to justify such regulation by trying to determine the value of industrial enterprises independently of their future net earnings is entirely futile. The fact that regulation changes and determines the value of industrial enterprises must be acknowledged; and the necessity and justice of the regulation of the prices of monopolized products must be established without denying patent facts.

The method of valuation proposed by the writer is substantially that which is generally followed in valuing real estate; here, however, it is more difficult to ascertain reliably the terms of the sales taking place, since they are not published like the transactions on the stock exchanges. The valuer of real estate takes the known sales, compares

Mr. Mayer.

the present and the estimated future net earnings of the properties sold, and determines that rate of interest which makes the value of the net earnings equal to the price obtained. He then ascertains the present and the estimated future net earnings of the property, the value of which is sought, and calculates, with the same rate of interest, the present value of all its earnings; this is the value of the property. The cost of production cannot be used where old properties are concerned, and especially where the property considered is of an uncommon kind, as practically all complicated industrial plants are.

The proposed method of regulation of prices of monopolized products uses only the sum of the values of a large number of plants manufacturing one product and the sum of their dividend and interest payments to the security holders, and then determines the price of this product; so that the average profit in this industry as thus determined is the same as that in competitive industries at the same time and in the same places. If the same method is used in valuing competitive and monopolistic enterprises, the errors in both, if the principle of the method is correct, will generally be of the same kind, and individual errors in both directions will largely balance, so that no injustice results.

When, as prescribed by some State laws, the net earnings allowed to individual enterprises are measured by their estimated value, then it is of the utmost importance that every single estimate is correct, and, in this case, every enterprise has a strong incentive to have its plant valued highly. In new States, where one has to deal only with new enterprises, a just valuation can be obtained by taking as the value of every business the amount of money paid in; and a just regulation can be based on values thus obtained. For old States the paper proposes one valuation based on average market values for the last few years. All future values would then be obtained from this first and only valuation plus the amounts of capital paid in thereafter. The value of all future enterprises, for the purpose of rate-making, would be determined by the capital paid in. The fluctuations of the stock market would be ignored and would have no influence on the prices of monopolized products.

Justice.—Hazy and fluctuating ideas of justice are the main causes of confusion in arguments on economic questions.

Justice consists in the establishment and maintenance of such relations among men as will secure the welfare of man. Man wills to live; that which furthers life is useful. Compensation in proportion to usefulness will secure the largest amount of usefulness; it will, therefore, most further life, and thereby will secure the welfare of man. By compensation in proportion to usefulness will the most useful be given the means of multiplying their kind and the useless

prevented from so doing. Only thus can an efficient and vigorous society be secured and maintained. Mr. Mayer.

The use of wealth as a tool for the production of wealth greatly increases the productivity of labor; the man who uses his wealth in this manner, therefore, is justly entitled to the increase of product secured thereby, and such increase is measured by the current rate of interest for safe investments.

The return secured by the use of capital varies greatly with the degree of industrial skill of the management selected and maintained in power by the financially responsible owners. Skillful owners and managers secure a much larger return on capital invested than the current rate of interest. Their skill creates this larger return under competitive conditions. This increase of return is measured by the profit, and owners and managers are justly entitled to the profit and loss of the management. Without this compensation skillful management would be but seldom obtained. To deprive skillful owners and managers of the profit they secure, under competitive conditions, would require the entire abrogation of present methods of selecting men for responsible control of all industrial enterprises. To abrogate profit, by limiting the return on individual enterprises to a fixed rate on the capital actually invested, would remove all inducement to efficient management. It would require such detailed supervision of all enterprises by the public as to amount practically to public management.

Extremely variable profits according to degree of skill of management are absolutely essential to efficient private ownership and management of industrial enterprises, and it is certain that any regulation attempting to enforce moderate uniform profits will lead to utter inefficiency of management and ultimately to the abandonment of regulation or of private ownership, and the introduction of public ownership and management. The control of all the important acts of the directorate must be placed with those who must bear the consequences of good or bad management. With uniform assured profits of individual enterprises the public bears these consequences, and must have full control and the appointment and discharge of the managers. The State cannot give to stockholders control and assured dividends; it can and should, however, do all in its power to protect them from being robbed by their directors and managers, by enforcing publicity of accounts, securing minority interests, and enabling them to form a correct opinion of the ability and honesty of their directors and managers.

A reliable directory of presidents, directors, and general managers, giving the rate of return secured by stockholders in the enterprises with which they were connected, would do much to secure the rapid

Mr. Mayer. advance of good and the removal of bad managers and directors from influential positions.

Monopolies.—A clear idea of the nature of natural monopolies is next in importance in order to define the problem.

A natural monopoly arises when the advantages secured by production on a large scale, as compared with production on a small scale, become so important that the existence of a single producer for a product or service on a given market offers great economic advantages over that of several producers.

Transportation of freight on a large scale can be furnished by a railroad at one-twentieth the cost of transportation on a small scale by wagons on public highways. For local traffic, therefore, railroads are natural monopolies. For through traffic, where the amount much exceeds the capacity of a double-track road, two or more competitors are economical, but the number of competitors for securing the lowest cost of production is so small that agreements to maintain prices and to stop competition become practicable and extremely profitable to all the railroads concerned. Competition, therefore, disappears as a regulator of prices, and public regulation is necessary to secure justice.

The disappearance of competition is due to the cheapness of large-scale production. If there is no public regulation of prices, the capital invested in natural monopolies often brings much larger returns than that invested in competitive enterprises. This generally leads to extensive stock watering, by promoters and reorganizers, the profits of which often go to them and not to the investors. If such stock watering were prevented by the honesty of the management, or by publicity of accounts and public supervision of capitalization, and its limitation to actual investment, it would lead, without regulation of the prices of the products or services furnished, either to excessive dividends or to uneconomical management or both. The excessive profits obtained by production on a large scale, without competition, are not the result of the special skill or industry of the management; they go with competition to the public, and should be made to go there by the regulation of monopolies where competition disappears. This should be accomplished without robbing innocent investors and without destroying the efficiency of private management.

The present values of monopolistic enterprises are largely ascertainable from actual transactions of sale in the security markets, and are the results of estimates of the present value of future net earnings under the regulation which is now considered most probable and most in harmony with the present preponderant sense of justice. Where these values are excessive and unjust, the injustice is due to the past erroneous conduct of the community toward monopolistic corporations, and the community—not individuals selected by chance—should be

made to bear the consequences of its past errors. To destroy or to revolutionize radically these values by the method of regulation adopted would be retroactive legislation, and, whatever legal maxims may be quoted to justify such a course, it would be robbing one class of investors for the benefit of others no more deserving. Such a course contradicts the fundamental principle of justice of compensation according to usefulness, and is thereby condemned. The new principle of regulation must be introduced with the least practicable injustice to past investors, and can only be applied strictly to all future investments. Justice, therefore, requires the method of valuation proposed in the paper. Mr. Mayer.

To proceed to various special objections: The formula for ascertaining the average profit of an old enterprise is criticized as not being correct, because the rate of interest on the left side of the equation should be the legal or a fixed rate. To make the formula absolutely correct this rate of interest should be the actual one at the time and place considered. The rate used in the writer's formula is certainly closer to this than a legal rate generally uniform over a whole State and representing what was desired by the majority of the legislature, not what existed at the time of its enactment. The same critic proposes to omit interest altogether from the left side of the equation. This would introduce a serious error, would exaggerate the profits in enterprises which take a long time to develop, and would relatively under-estimate the profits in enterprises with a quick return. A regulation based on such omission of interest would be unworkable, because no capital could be secured for enterprises of slow development. There is no circular argument in the paper, and such a statement results from careless reading. Any regulation of prices requires the use of mathematics; to avoid this use, therefore, is impossible.

The purpose of the chapter entitled "Charging What the Traffic Will Bear" is to show in a general way why, and to what extent, monopoly prices are higher than competitive prices. The context shows that what is intended is to compare the prices under monopoly and competition with the same scale and method of production. The small change in quantity of demand which is considered in the argument will not produce so large a change in cost of production as to affect it materially. The writer believes that the inference drawn in this argument is substantially correct, though not required for the regulation proposed.

A mere observation of the relevant facts shows that the unregulated monopoly prices are generally too high and should be reduced by just regulation. Monopoly prices are only lower than intelligent competitive prices when the monopolists do not pursue their interests intelligently, as, for example, when they engage in the production of a service or product, the degree of usefulness of which does not permit

Mr. Mayer. a price which would give an ordinary rate of profit; this case, however, is expressly excluded from the argument.

It is often economical to have only one producer for a local product. In this case, also, the competitive price—that is, the price giving to average ability average competitive profit—is lower than that charged by an intelligent monopoly pursuing only its own financial interest without fear of regulation.

The question has been asked: How would this regulation proceed if there is only one producer?

The excessive concentration of ownership and management of many disconnected enterprises is largely due to the effort to obtain monopoly prices. If competitive prices are enforced for monopolized products, it will not pay to concentrate ownership by buying up independent enterprises to a larger extent than is justified by the savings in cost of production thereby secured. The day of single ownership of whole industries will be thereby postponed indefinitely and need not concern us at present.

A frequent objection to the establishment and usefulness of such commissions, made privately, is based on experience with existing commissions, which is sometimes disappointing.

Sometimes, such commissions are not composed of men of sufficient ability, and they often work under impracticable laws. If these laws were strictly enforced, they would cause so serious a disturbance of the value of investments that the commissioners would be discharged. As a necessary result, little is done. Often, with otherwise reasonable laws, the funds available are entirely inadequate for establishing a strictly scientific regulation. One of the most successful commissions, though it has inadequate powers, is the Interstate Commerce Commission. It has been guided approximately by the principles advocated in the paper. It has regulated the rates without causing a regulation in security values. It has not attempted to secure a uniform rate of profit in different enterprises; it has endeavored to establish such rates as would enable the railroads to secure new capital for extensions and improvements in service, or such rates as would make the average profits on capital thus invested the same as in competitive enterprises. Railroads are for through traffic, which is largely competitive. With those rates which are the result of real competition the commission has interfered very little. One important result of competition, among enterprises in which the rate of profit largely depends on the amount of business done with a plant of large capacity, must here be mentioned. When there are several such independent competitors for a total amount of business far below their combined capacity, one of them, by reducing prices, may attract so much new business that his profits are increased by the reduction. The others, by their losses, may be induced to reduce prices still further in order to recover their

customers. This may go on, with a drop in the value of the securities of the competitors, and may result in their consolidation, either before or after the bankruptcy of some of them. Such competition inevitably results in its elimination and in gross injustice, during the process, to many of the security holders. Among railroads, this is largely a phenomenon of the past, and is now prevented, either by gentlemen's agreements or by the just interference of the legislatures and the regulating commissions. Such unreasonably low rates do not give compensation according to usefulness; they are equally unjust with excessive rates, and, without regulation, inevitably result in unreasonably high rates thereafter, since they frighten away capital from enterprises in which they occur. Mr. Mayer.

The laws establishing regulating commissions should demand such regulation of the prices of monopolized products as will yield equal average profits in competitive and monopolized industries at the same time and place. They should prescribe that the present value of each enterprise be determined by the same general principles as are used in the valuation of real estate. Future values for rate-making purposes should be determined from the present value plus the additional capital paid in by purchasers of newly issued stocks and bonds. The dividends and interest paid out should be considered as the profit of the owners. This regulation does not require any impracticable or very expensive valuation or determination of profits; it does no injustice to any class of present investors or to the consumer; it does not destroy the efficiency of private management, and will secure the same average profit for future investors in competitive and in monopolized enterprises. The only practicable alternative to such regulation is public management and ownership of all natural monopolies. If public ownership is established, similar commissions will be just as necessary as with private ownership. The efficiency of management of any industrial enterprise can only be determined by a scientific comparison of its costs of production with those of other enterprises producing similar services or goods. Such a comparison cannot be made by the voter or by any individual; it requires the systematic investigation of all similar enterprises of a large territory.

Competition, where it exists and where it can be economically preserved, automatically removes uneconomical, and increases the scale of operation of the most economical, managements. For public management this function must be performed by qualified commissions with equal relentlessness in order to secure a tolerable degree of efficiency and enterprise. The inefficient regulation of the prices of monopolized products has been largely responsible for the introduction of public ownership of industrial enterprises. If this regulation is of such a kind as to destroy the efficiency of private ownership, public ownership is the necessary result. Without harm to economy and enter-

Mr. Mayer. prise, the public regulation of private enterprises may prescribe minimum wages, healthy conditions of work, compensation for accidents and sickness, old age pensions, and protection of women and children; it may prescribe the quality of the products or the services rendered; and it must prescribe the prices for monopolized products and services.

The writer's aim has been to describe a method for the regulation of the prices of monopolized products and services which would make public ownership and management superfluous. Why should we fear public ownership and management? The public has but average intelligence. All improvements in production are first conceived by exceptional intelligence. The mind which originates an improvement needs a capitalist able to appreciate his invention and to introduce it into practice. If, by the prevailing industrial system, you assure to the capitalist the savings in cost of production obtained by the improvement until the new method has become common, then the inventor of a real improvement has a reasonable chance to get it introduced. With public ownership, the inventor of an expensive improvement must, for securing its introduction, first convince a large number of mediocre men who will each gain little by its adoption. Public departments, therefore, are unduly conservative, and progress is slower than with private ownership.

A board of directors composed of large stockholders is more likely to watch closely the economy of management and to select the chief executive officers with the exclusive aim of finding the best man, than a commission appointed by a mayor or elected by the voters. The chief executive officers of a stock company know that their efficiency will govern their tenure of office, and all the employees will be selected, retained, advanced, and discharged mostly by efficiency, while, thus far, considerations other than efficiency often have more influence in public than in private enterprises.

An inefficient private enterprise working under either natural competition or with prices regulated by an efficient commission will, at present, be more rapidly put out of business than an inefficient public department. Some better means than now available, however, are necessary for making a reliable comparison of the efficiency of private and public enterprises and for correcting erroneous opinions and consequent mistaken action in choosing either public regulation or public ownership. Thus far, competent public commissions are the only bodies that can efficiently perform this work.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

A FOUR-TRACK, CENTER-BEARING, RAILROAD DRAW SPAN.

Discussion.*

By Messrs. Howard J. Cole, L. J. Le Conte, and Louis H. Shoemaker.†

Howard J. Cole, M. Am. Soc. C. E.—The speaker desires to call attention to the fact that the first four-track draw-span in America was built over the Harlem River in New York City, about fifteen years ago, by the New York Central and Hudson River Railroad, Walter Katté, M. Am. Soc. C. E., Chief Engineer. Mr. Cole.

It has three trusses, 389 ft. from center to center of end pins and 26 ft. apart in the clear, carrying the four tracks on a ballasted floor.

There was no limiting depth, in this case, between the base of the rail and the top of the masonry, but the United States Government required 24 ft. clearance above high water. This draw-span is notable for its weight, 2 500 tons.

L. J. Le Conte, M. Am. Soc. C. E. (by letter).—The writer is much pleased with this new design for a center-bearing draw-bridge, particularly because, for many years past, he has had a strong prejudice against all center-bearing devices. Mr. Le Conte.

The general scheme of doing away with the old-fashioned center tower, and bringing both sets of main inclined posts to one point over the center of the center pier is commendable. The heavy compound box-girder, which takes the place of a floor-beam at this site, is designed to carry the entire dead load of the bridge—some 1 400 tons—when swinging open. This is all simple enough, but when it comes to supporting this heavy compound box-girder on a center-pintle bearing, the serious side of the problem is brought boldly to the front.

* Continued from May, 1912, *Proceedings*.

† Author's closure.

Mr. Le Conte. The writer regrets that the plans submitted do not show more of the useful details, as that would probably permit of fair criticism of the design; nevertheless, the details are sufficient to indicate a doubt as to the ability of the design to stand up permanently under the dead load strains when the draw-span is on the swing. The desired details are lacking particularly in the case of the small cross-girders and eye-bars, which are designed to carry the entire dead load, also, the packing of the joints and girder connections, all of which it is very important to know in order to make fair discussion possible. Of course, it is natural to presume that the design is all right in its details, but the plans ought to show enough to establish the fact with reasonable certainty. It is very important that all parts of the compound box-girder and the little overhead cross-girders should be designed so as to be easily inspected and painted at any time that it may be advisable or necessary.

The center-bearing device, of course, has one great advantage which cannot be over-estimated, namely, the ease with which it turns. If this design proves to be a success, it will be noted as an important step in the right direction.

Mr. Shoemaker. Louis H. Shoemaker, M. Am. Soc. C. E. (by letter).—As Mr. Le Conte expresses a doubt as to the possibility of supporting the load of 1 400 tons in a permanent manner by the method described in the paper, the writer submits Fig. 3, a detail of the center casting and longitudinal supporting girder, which he trusts will explain the construction and remove all doubts as to its sufficiency.

By distributing the load over four transverse girders, it was possible to keep the flange sections and rivet grips well within the limits of good practice. This arrangement also adapted itself to the best possible device for supporting from the center casting, that is, with eye-bars and pins, by which a practically perfect distribution of load through girders and hangers is secured.

Attention is called to the superiority—in the above respect as well as in that of accessibility—of this design over the usual method of supporting heavy draw-bridges from center bearings, in which two transverse supporting girders, with bolt girders framed between them as close as possible to the center casting, are hung by a number of large round rods from a broad saddle girder. The design of the longitudinal supporting girder, *C*, was rendered comparatively easy by taking the necessary depth, and it was made sufficiently wide to admit of painting.

Mr. Shoemaker.

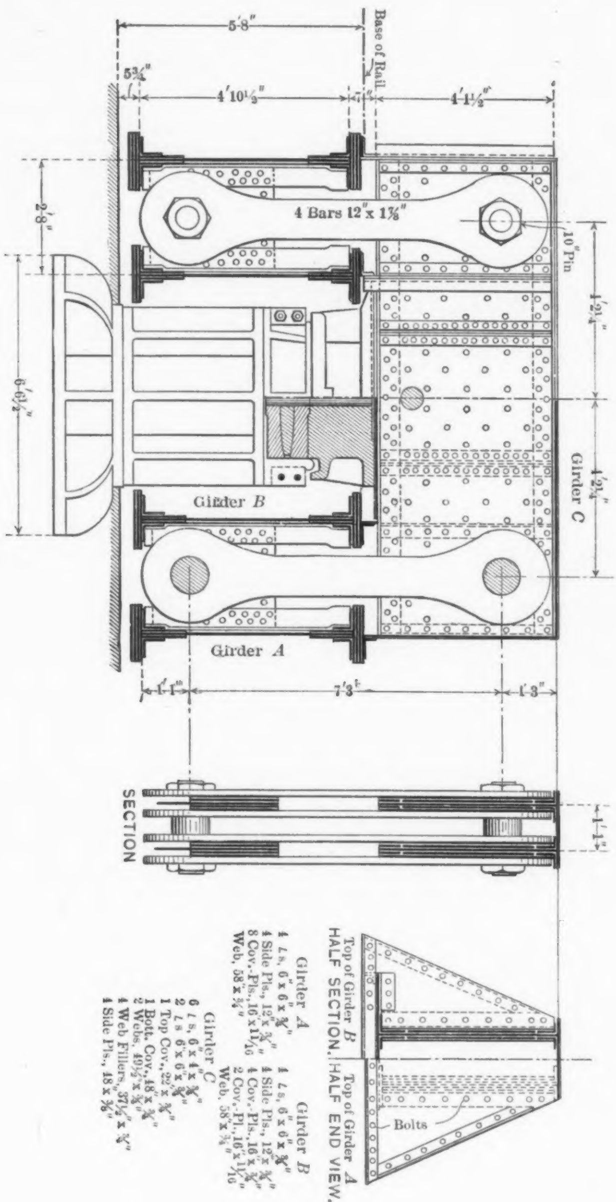


FIG. 3.

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PAPERS AND DISCUSSIONS

A REINFORCED CONCRETE INFILTRATION WELL AND PUMPING PLANT.

Discussion.*

By Messrs. H. F. Dunham and Frederick N. Hatch.†

H. F. Dunham, M. Am. Soc. C. E. (by letter).—There are excellent features in this brief paper. The clear statement of facts relating to the construction, and the included costs, with efficiency data, are valuable contributions; but further information relating to the source and volume of supply would be of interest. Mr. Dunham.

When a well is sunk beside a river it is often difficult to prove that a large proportion of the supply comes from the river. If the sand and gravel beds penetrated by the well are extensive, it may be suspected, and in some cases it has been proved, that during a greater part of the time, and especially when the river is falling, the supply is water intercepted from land areas on its way to the river. Instances are recorded where a similar well, its connected machinery not being in operation, received quantities of water by overflow at the top. In such cases the river water finds a short cut—an artificial channel—through the well to partly exhausted gravel or sand beds which would be filled more slowly under natural conditions.

Wells supplied in large part from the land side are more satisfactory in one particular, for the tendency of silt to fill the interstices in the sand between the river and the well or near the well is lessened. Records should be kept showing the difference in elevation between the water level in the well when certain definite quantities are being supplied and the extreme low water in the river at that time. Year by year the record indicates the extent of the silting up of the beds.

* Continued from May, 1912, *Proceedings*.

† Author's closure.

Mr. Dunham.

Two or three changes may be suggested in the design to insure the following advantages:

1. Provision for increasing the supply and for restoring the original conditions if the present supply diminishes.

2. A reduction in the amount of electrical energy lost in friction.

3. Better opportunities for attention to machinery and for repairs.

By these items attention is drawn to estimates for a modified structure or housing in place of the cylindrical well, a regular intake supply main of proper size, and displacement pumps run by electric motors. The foundation for such a structure would be but little below low water.

The supply main, at about the same elevation, should be laid on a slightly descending grade, with branches to connect with vertical strainer wells which could be shut off for cleaning, repairs, or for an extension of the system when necessary.

The pumps should be set with their discharge valves below the level of the intake, and should be under automatic control. Then all interior parts would be accessible without the use of ladders, and there would be no trouble from water. The friction or loss of energy would be reduced from its present 70% to 25% or less, thus reducing the cost of electric current by from $300 to $400 per year, at a low estimate per kilowatt. The first cost would be differently apportioned to the various parts, but the total should not exceed the cost figures given by the author.

Mr. Hatch.

FREDERICK N. HATCH, JUN. AM. SOC. C. E. (by letter).—Mr. Copeland and Mr. Dunham have brought out a few points which were not considered at length in the paper on account of a desire to confine the matter to a description of a rather novel method of constructing a well for obtaining a relatively small supply of water. The fact that these points were not taken up in detail must not be considered as an indication that they were not fully considered at the time the plant was designed.

The statement that the well "would receive water by infiltration from the river through the intervening sand and gravel" was not intended to convey the idea that the flow from the river to the well would take place by direct filtration along the shortest path at all times; during high-water stages of the river, it was expected that a large part of the water entering the well would come from the surface flow of the river by direct infiltration, and prolonged observation since the well was completed seems to indicate that such is the case. It was fully realized that such direct infiltration would not take place in a marked degree during low-water stages, but it was expected that a certain amount of the surface flow would reach the well along with the subterranean flow, because of filtration through the bed of the stream at some more or less distant points.

In regard to the impurities in solution in the water, it may be of interest to note the difference in the quality of the surface flow of the river and the true ground-water at this point, as indicated by analyses of two samples, taken at the same time and analyzed by the same chemists. The results are given in Table 1. Mr. Hatch.

TABLE 1.—PROBABLE CONSTITUTION OF THE INCRUSTING SOLIDS.

In grains per United States gallon.

	Sample No. 5. from a test well after 24 hours' pumping.	Sample No. 6. from the river.
Silica	0.70	1.46
Alumina and oxide of iron	0.13	0.34
Carbonate of lime	11.70	1.40
Carbonate of magnesia	2.60	1.17
Sulphate of lime	2.30	5.86
Sodium chloride	0.50	1.90
Total solids	17.69	13.45
Hardness calculated as calcium carbonate	14.38	6.38
Reaction	Alkaline	Alkaline

As to the danger of the inflow being gradually reduced by the silting up of the interstices in the surrounding sand, it may be said that such a possibility was recognized, and it would in no way prove serious in this case, as it would then be only necessary to place a crib in the river and make a direct intake to the well with cast-iron pipe. The total cost of the plant, including such an intake, would not exceed the cost had the well been located originally where it would receive water directly from the river, and, until such a condition causes trouble—if it ever does—the water obtained is of better quality than would be received if it were taken directly from the river, and the fixed charges on the investment are less.

In considering other methods of operation of the pumping plant than the one selected and described in the paper, it must be borne in mind that the water level of the river may vary as much as 69 ft., and that the banks are submerged at high-water stages. Any pump housing would be required to provide for those conditions. If electrically-driven displacement pumps were set a little below low water—as the writer understands Mr. Dunham to suggest—the housing of them would be a more serious problem than was the housing of the centrifugal pumps, because of the necessity of keeping the driving motors dry. Mr. Dunham states that with his scheme there would be no trouble from water; but it should be noted that all his vertical strainer wells would be inaccessible for many months of the year on account of the water of the river standing over their tops.

Mr. Hatch.

It is not clear how Mr. Dunham proposes to make such a large saving of power over the present arrangement, or how he arrives at the figure, 70%, which he states to be the loss of energy at present. It is obvious, however, that, with the same electric power, an electric motor driving a centrifugal pump will be just as efficient as one driving a displacement pump; it is also apparent that the energy required to overcome the static pressure and friction head in the discharge pipe will be the same in either case. The only remaining source of lost energy is in the pump, and the writer does not know of any type of displacement pump which would show an energy loss of only 25% where the loss with a centrifugal pump would be as much as 70 per cent.

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PAPERS AND DISCUSSIONS

THE APPRAISAL OF PUBLIC SERVICE PROPERTIES AS A BASIS FOR THE REGULATION OF RATES.

Discussion.*

BY MESSRS. WILLIAM BROKAW BAMFORD, JAMES V. OXTOBY, CHARLES H. HIGGINS, HENRY FLOY, J. MARTIN SCHREIBER, WILLIAM J. BOUCHER, R. D. COOMBS, A. H. VAN CLEVE, AND W. KIERSTED.

WILLIAM BROKAW BAMFORD, M. AM. SOC. C. E. (by letter).—It is axiomatic, for the correct application of the principle set forth by the author, or for the equitable appraisal or valuation of any property, that the "expectancy" or probable life of the property be determined with reasonable limits. The writer is contemplating the presentation of a paper on the probable life or endurance of property, and, for the purpose of this discussion, will point out only certain general principles. Mr. Bamford.

The "expectancy" or "actual life" of any property or individual can only be determined by summarizing the probable "endurance" of the various elements which may affect in any way the "actual life." For a property, as for an individual, the result at best is but an approximation. Nevertheless, it is possible to tabulate facts obtained over a series of years so that tables of the probable life of a property can be prepared with as reasonable a degree of accuracy as those which determine the probable life of the individual.

Until methods for properly forecasting the actual life of property are put on as stable a basis as the preparation of mortality tables for human life, we will have difficulty in adjusting equitably and

*This discussion (of the paper by C. E. Grunsky, M. Am. Soc. C. E., published in *Proceedings* for April, 1912, and presented at the meeting of June 5th, 1912), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Bamford. scientifically, the various financial questions connected with public service as well as private property.

For the purpose of clearness in discussing the subject, the writer has preferred to devise and use the term "endurance" rather than "actual life," "expectancy," or "depreciation."

The "endurance" of a property may be said to be its power and ability to prolong its life or existence against the adverse forces or influences of any kind which threaten it. It is its power to remain in the same state without perishing. The endurance may be considered as being ultimately established by the "actual life" of the property.

It is indisputable that if a property has no endurance it cannot last, and that its endurance is due to various factors which tend to prolong or shorten its life. The elements which influence the endurance of a property might be divided into the physical causes which threaten its existence as a structure (depreciation or deterioration) and the various economic or commercial causes which threaten its life as a property (obsolescence). In addition, there are certain hazards which threaten its life, which might be summarized as follows:

(*A*). Physical Endurance of Property (depreciation or deterioration)—threatened by:
- (*1*) stability of structure;
- (*2*) physical deterioration;
  - (*a*) in materials composing structure,
  - (*b*) in methods of construction.

(*B*). Economic or Commercial Endurance of Property (obsolescence)—threatened by:
- (*1*) obsolescence due to improvements in machinery, processes, etc.;
- (*2*) obsolescence due to lack of serviceability for use;
- (*3*) obsolescence due to changed industrial and social conditions;
- (*4*) actual cost too excessive for present value.

(*C*). Hazards to Endurance of Property:
- (*1*) destruction by fire;
- (*2*) destruction by act of God—earthquake, cyclone, flood, etc.;
- (*3*) destruction by domestic violence or foreign wars.

Thus it will be seen that "endurance" or actual life is threatened and controlled by elements composing the three divisions of (*A*) physical endurance, (*B*) economic endurance, and (*C*) hazards. The actual life, therefore, will be terminated by the element which has the least amount of "endurance." In a rapidly growing community and progressive age, economic endurance is usually shorter than

physical endurance. In the present age evidences of this are seen on every hand, obsolescence terminating the actual life of property the physical endurance of which may still be of "indeterminable duration." Mr. Bamford.

All questions of depreciation, as usually considered at the present time, should be resolved into attempts to establish a standard for the endurance or actual life of the property. In the majority of engineering investigations of the endurance of property, however, the primary efforts are directed toward solving the endurance of the physical or structural elements, to the neglect or subordination of the economic or commercial elements and hazards.

Neither the physical nor the economic endurance can be considered alone in determining the actual life of property; both must be determined, together with the hazards, and that one having the least endurance will be the one to govern the case in question. The probable physical life is no guide to the probable economic life; while the actual economic life is a positive check to the actual—not the potential—physical life.

It is true, of course, that the probable economic life is more difficult to determine accurately in advance than the probable physical life; but, as with all forms of insurance, it is possible to apply the law of averages so that the resultant will be equitable to the public service companies as well as to the public.

In its practical application it would be advisable to have a State commission to establish uniform standards for the determination of probable economic life and "endurance," which should be subject to periodic adjustments. Such a procedure is not novel, as the English Local Government Board has undertaken just such a work in establishing periods for the redemption of authorized bonds for local improvements.*

Without the possibility of establishing the "expectancy" or "endurance" of property equitably, the very admirable methods proposed by the author will prove valueless. It is hoped that in the future more attention will be given to "endurance," rather than to concentrating efforts solely on physical "depreciation."

JAMES V. OXTOBY, ESQ. (by letter).—The writer has read this paper with interest, and wishes to express his appreciation of the work which Mr. Grunsky has done toward making clearer the facts regarding the element of depreciation in making appraisals of public utility properties. Mr. Oxtoby.

It is conceded that an owner of a public utility is entitled to earn a reasonable return on his investment. He must also receive from the business, on its being wound up, or on its sale, an amount equal to the principal of his investment. If depreciation of plant is

* *Engineering News*, Vol. 54, p. 462.

Mr. Oxtoby. provided for on a sinking-fund basis, the interest which the depreciation fund earns is part and parcel of that fund, which must be added to it annually in order to bring it up to the required amount at the end of the depreciation period. Otherwise, the fund becomes at once impaired. The annual amount paid into the sinking fund by the business is smaller than it otherwise would be, because it is expected that the earnings of the fund will be added to it, and that, by compounding the interest yearly, the fund, at a predetermined date, will equal the amount of the depreciation. If the owner uses any part of this fund, whether principal or interest, he is really using part of the principal of his investment.

In appraising a plant for rate-making purposes, its value is its reproduction value, and not its reproduction value less depreciation. The entire depreciation must be earned from the public. If the sinking-fund method is used in computing the amount of the yearly depreciation, it is apparent that the fund must exist and must be made to earn its own increment. Some students of this question have stated that the depreciation fund has already been earned from the public, and that including the depreciation fund in the valuation would be requiring the public to pay dividends on moneys already paid in by it. This is not true where the sinking-fund method is used. By that method the public is asked to pay, not the full amount of the yearly depreciation, but only a part thereof, the compound interest earned on what the public has so paid making up the deficiency.

This confusion of ideas seems to result from the failure to distinguish between the repayment and withdrawal of investment from year to year, and the establishment of a sinking fund which does not mature until some date in the future. In the first case the investor has got back his money and can apply it to other uses. In the second case his money is withheld from him, being locked up in the sinking fund; and he is not only unable to apply it to other uses, but must see to its being invested in absolutely safe securities not affected by depreciation, in order that at maturity the fund may be adequate to meet the liability for which it was provided.

A similar confusion of ideas affects many students when they are called on to state a fair price for a utility expropriated as a going concern. They err in stating the price as reproduction less depreciation, the vendor to retain the depreciation fund. In practice the depreciation fund is locked in securities which may or may not be then marketable for their assumed value. The law authorizing expropriation should impose on the purchaser the burden of marketing these securities promptly, or of purchasing them himself. Otherwise, the investment made by the vendor is not fully released. In this the writer intentionally omits reference to the equitable requirement

of a higher price in expropriation proceedings than in rate-making proceedings. Mr. Oxtoby.

Assume an investment of $25 000 made in a plant, which, at the end of twenty years, will have a depreciated or junk value of $5 000. Assume 6% as fair return on the plant investment and, for convenient figuring, assume that it is possible to invest a sinking fund in safe securities paying 6 per cent. Table 12 is based on these assumptions, and on the further assumption that the sinking-fund method of calculating depreciation is correct.

TABLE 12.

Year.	Value of plant at end of year, by assumption.	Return of 6% on $25 000.	Annual depreciation sinking fund, 6% basis.	Interest on fund at 6%.	Total depreciation fund at end of year.
0	$25 000.00	....	....	....	....
1	24 456.31	$1 500.00	$543.69	....	$543.69
2	23 880.00	1 500.00	543.69	$32.68	1 120.00
3	23 269.11	1 500.00	543.69	67.20	1 730.89
4	22 621.58	1 500.00	543.69	103.85	2 378.42
5	21 935.22	1 500.00	543.69	142.70	3 064.78
6	21 207.60	1 500.00	543.69	183.88	3 792.40
7	20 436.37	1 500.00	543.69	227.54	4 563.63
8	19 618.82	1 500.00	543.69	273.81	5 381.12
9	18 752.46	1 500.00	543.69	322.86	6 247.54
10	17 834.16	1 500.00	543.69	374.85	7 165.84
11	16 860.52	1 500.00	543.69	429.95	8 139.48
12	15 828.46	1 500.00	543.69	488.37	9 171.54
13	14 734.48	1 500.00	543.69	550.29	10 265.52
14	13 574.86	1 500.00	543.69	615.93	11 425.14
15	12 345.07	1 500.00	543.69	685.50	12 654.93
16	11 042.09	1 500.00	543.69	759.29	13 957.91
17	9 660.93	1 500.00	543.69	837.47	15 339.07
18	8 196.90	1 500.00	543.69	920.34	16 803.10
19	6 645.02	1 500.00	543.69	1 008.19	18 354.98
20	5 000.00	1 500.00	543.69	1 101.33	20 000.00

From the foregoing it is evident that if the owner is required to accept as profit less than $1 500 yearly, he has been deprived of a reasonable return on his $25 000 investment.

A plant probably could not continue to operate usefully without replacement before the end of the depreciation period, but this illustration, taken with the many which Mr. Grunsky has furnished, demonstrates the principle that if the annual depreciation is computed on a sinking-fund basis, it is inequitable to use depreciated value alone as the basis for the reasonable return to which the owner is entitled.

If the depreciation is computed on the straight-line basis, the interest which the depreciation fund earns is really an earning. If depreciation thus computed is paid to and withdrawn by the owner, he is entitled to earn his fair return only on the remaining value. The owner of a public utility, however, is bound to keep up the plant to a high degree of efficiency and must rehabilitate it when necessary, which practically precludes withdrawal.

Mr. Oxtoby. Mr. Grunsky demonstrates that if the depreciation fund is used for replacements, which themselves at once commence to depreciate, the sinking-fund method is impracticable. The fact is that depreciation is not an exact quantity, but must be determined by the exercise of fair judgment. Its amount at any time is the difference between actual investment and present value. It is highly improbable that, during a series of years, the actual depreciation of a plant will coincide with the accumulation of money by a sinking-fund rule. A public service plant should be maintained from its earnings so that its present value will always equal its original cost. If not, the difference should be in a depreciation fund, by whatever method it is computed; and a fair return should be computed on the total value which serves the public, whether this value is in plant or in depreciation reserve. Real earnings by the depreciation reserve are a part of the earnings of the plant. The increment, however, of a fund which is accumulating on a sinking-fund basis is not an earning.

In this discussion the writer has, in general, merely stated in his own way ideas which appear in the paper. His appreciation of the argument contained therein has been his excuse for doing so.

Mr. Higgins. CHARLES H. HIGGINS, M. AM. SOC. C. E.—This paper should receive the careful attention of the members of this Society, for it deals with matters far more fundamental than a reading of the title or introduction might lead one to believe. In fact, no where has the speaker found the main issue stated clearly, but a student of these matters cannot read this paper carefully without finding the author constantly returning to this undefined issue. Whether it be veiled purposely or merely clouded because not clearly seen by the author, it cannot escape being the point of such overwhelmingly importance in this paper as to challenge attention.

On page 398* occurs the following sentence:

"Perhaps the use of the term, 'value,' in this connection is unfortunate, because it is not clear why 'value,' as ordinarily defined (which is not always synonymous with capital reasonably and properly invested), should be made the criterion of allowable earnings."

And, again, on page 444:*

"* * *, and to the ruling of the Courts, which hold that owners of public service properties are entitled to a fair return on the 'value' of such properties.

"If it be found that the ruling of the Courts is not subject to modification, or, in other words, that appraisals must be 'value,' as 'value' would be determined by a purchaser, that is to say, for the tangible elements in most cases, cost or cost of replacement less depreciation, or something practically equivalent thereto, * * *."

**Proceedings*, Am. Soc. C. E., Vol. XXXVIII (April, 1912).

Mr. Higgins.

In these lines the real meat of the matter may be found, the author taking issue with the decisions of the United States Supreme Court.

Stripped of all verbiage, this paper deals then, not with the methods of making appraisals of public service properties under the existing law as interpreted by the Supreme Court, but with what, in Mr. Grunsky's opinion, the law should be. The latter may well be an equally proper matter for discussion before this Society, but it is certainly very different from the former.

To present this matter clearly, the speaker will illustrate. As early as 1898, in the leading case of Smyth *v.* Ames,* in the Nebraska maximum rates cases, the Supreme Court laid down the principle that the basis of all calculations, as to the reasonableness of rates, must be the fair value of the property used, and specified certain matters to be taken into consideration in ascertaining the fair value: the original cost of construction, the amount expended in permanent improvements, the amount of market value of the bonds and stock, the present, as compared with the original, cost of construction, the probable earning capacity of the property under the particular rates prescribed, and the sum required to meet operating expenses; all to be given such weight as would be just and right in each case. Justice Harlan was careful to add: "We do not say that there may not be other matters to be regarded in estimating the value of the property."

The following year, in the case of the San Diego Land Company *v.* National City,† the Court held "what the Company is entitled to demand in order that it may have just compensation is a fair return upon the reasonable value of the property at the time it is being used for the public." The Supreme Court, then, as early as 1899, had adopted present value as the standard, leaving undetermined how a reasonable value is to be ascertained and what constituted a fair return.

Again, in 1903, in San Diego Land and Town Company *v.* Jasper,‡ the Court said:

"It no longer is open to dispute under the Constitution that what the Company is entitled to demand, in order that it may have just compensation, is a fair return upon the reasonable value of the property at the time it is being used for the public."

In a masterly review of the subject of regulation of railway rates, Judge Swayzee of the Supreme Court of New Jersey, says:

"Novel questions of this character will arise with increasing frequency, and require the most careful consideration. Like most other

* 169 U. S., 466.

† 174 U. S., 739.

‡ 189 U. S., 439.

Mr. Higgins. questions in every department of law, they are in their origin rather questions of fact than questions of law, although in course of time the rules become settled and thus become rules of law. In their origin and as yet many are questions of sound business management and engineering science. The law prescribes reasonable return upon a reasonable valuation. What is a reasonable return and what is a reasonable valuation must vary with the circumstances of each particular case."*

It may be accepted then as an established rule that the appraisal of a public service property to be used in fixing rates should show the fair value of the property.

Now, Mr. Grunsky argues that for the fair "value" of the Supreme Court there should be substituted something which he calls "capital properly and reasonably invested," stating, as a fundamental principle, that:

"The valuation of a public service property and its earnings must bear such relation to each other that there will be returned to the owner, within the life of the property, the capital which he has properly invested in it, and in addition thereto, interest at a reasonable rate, upon such amount of capital as from time to time actually and properly remains in the property as an investment."†

In this Mr. Grunsky takes issue with the Fourteenth Amendment as interpreted by the Supreme Court since 1898.

The author's "fundamental principle" quoted above, if applied, as outlined in the paper, would cut both ways. In determining what is termed "capital reasonably and properly invested" according to the author's plan, earnings and dividends, or interest, are considered apparently from the beginning, and if these dividends have been less than what is determined as a reasonable rate of return, the difference is considered as remaining in the property as invested capital. On the other hand, in the case of past earnings above the determined rate, Mr. Grunsky speaks as follows: "It is possible, of course, in the case of large earnings in the past, that a portion thereof should be considered as capital returned to the owner." In other words, the author would appear to propose not only the determining by a State of a reasonable rate for the present and future, but actually to make it retroactive. In this, it would appear to the speaker, that the author might run up against the Constitution of the United States for the second time.

Mr. Grunsky seems inclined to treat all investors as if they were owners of State bonds. For a lucid explanation of the relative positions of the holders of different classes of securities and their relation to the public, nothing better in its way can be found than the Report

* *Quarterly Journal of Economics*, May, 1912.

† *Proceedings*, Am. Soc. C. E., Vol. XXXVIII (April, 1912), p. 438.

of the Railroad Securities Commission, of which President Hadley, of Yale University, was Chairman. Mr. Higgins.

To the speaker's mind, the Courts, in fixing on fair valuation as the basis for all calculations, have taken the only position economically sound.

The speaker accepts the conclusion of Judge Swayzee already quoted: "The law prescribes reasonable return upon a reasonable valuation. What is a reasonable return and what is a reasonable valuation must vary with the circumstances of each particular case." This states clearly the proposition before an engineer making an appraisal or valuation for use in the fixing of rates under the law as it exists to-day, and there are enough technical difficulties yet to be settled by engineers. The broader question dealt with in this paper is very interesting, but should not be confused with methods under the law as established.

Henry Floy, M. Am. Soc. C. E.—The speaker cannot agree with the views expressed by Mr. Higgins. Mr. Grunsky has brought out quite clearly a very important question, which must still be fought out and decided by the Supreme Court, and that is, the basis of "fair value." The various opinions rendered by the Supreme Court have not yet fairly and squarely determined the question as to whether or not "fair value" shall be taken as that derived from a consideration of accruing, theoretical depreciation, or something in addition to such value. The speaker is inclined to agree with the author that the investor is entitled to a return on his investment, or the cost of reproduction, if his property is kept in good operating condition. Present value accurately obtained from a consideration of accruing depreciation must vary from day to day, thus forming a fluctuating and impractical value on which to base rates or capitalization. Such fluctuating value is too unstable and unfair ultimately to be received and accepted for fair value, as the author has brought out. Mr. Floy.

It is very difficult, even for lawyers, to interpret the decisions of the Court, and surely it is much more difficult for engineers to ascertain exactly what these decisions of the Supreme Court which have been referred to—the Consolidated Gas case and the Knoxville Water case—mean in the last analysis. It would seem as if, by these decisions, the Court intended to convey the idea that something more than a theoretically depreciated value should be made the basis on which a return is to be allowed.

It is possible that, if "the recall" is established, some new decisions may be expected; yet, at the present time, we may feel confident that the Courts, regardless of how excessive the returns have been in the past, would maintain the position that present and future returns will be based on the fair value of the property. The Courts will not attempt

Mr. Floy. to reduce earnings in the present below the fair return on the fair value of the property, even though the owner for some years previously has been earning more than a fair return.

A return of 4% has been mentioned in the discussion, but the Supreme Court has never yet named anything less than 6%, and that for conditions such as exist for a monopoly in New York City. Of course, a fair return for any particular property depends on that property, and one like the Consolidated Gas Company in New York City—a monopoly in the greatest city in America—is not running as much risk in the way of securing a return as some other property in a small town in the West, and the Courts and Commissions have recognized this fact. The Commission of New York City, for example, has in certain decisions explicitly stated that certain gas and electric corporations were entitled to a return of 7½ or 8% on the total value of their properties. This means that the stockholder may obtain a very much higher rate of interest for his holdings, while the bondholders, who have a prior lien on the property, are willing to accept a 4, 5, or 6% return for their share of the investment.

In the matter of valuing real estate, the Courts have quite generally held that, in determining present value, the owner is entitled to appreciation, and this theory was distinctly enunciated in the Consolidated Gas case, where the U. S. Circuit Court approvingly quotes the language of the Supreme Court in another case to the effect that "the value of the property at the time it is being used" should be taken.

It must be recognized that fair values may be different for different purposes. The property of a corporation which is to be assessed for the purpose of special franchise tax in New York City, for example, would have a different value from that computed for other purposes, because a franchise tax relates only to the property in the street, which may be a relatively small part of the total property of the corporation. The value of property for rate-making purposes, in a similar way, may not be the same as that for capitalization, because a part of the property may be held simply as an investment or in connection with an allied business, so that the value for rate-fixing purposes, in such case, would be quite different from the value proper for capitalization. These questions of "fair value" are of comparatively recent origin, and are by no means easy of solution. Until a few years ago, no one appreciated or considered the fact that the operation of public utility property was a matter of much public concern. Public utilities were given grants and encouraged to make investments with the expectation of large returns, certainly 8 or 10%, possibly 15 or 20%; but lately, as the corporations have developed and become in many instances monopolies, either through crushing out competition or buying up their competitors, the public has been compelled to deal with the

situation on a basis radically different from that formerly allowed. This has resulted in the development of what may be called a theory, which we are attempting to make a science, with regard to the control and operation of public utility properties, so that the situation is quite different from anything that existed fifteen or twenty years ago. To-day we look upon a public utility corporation as entitled to special privileges, such as the use of public streets, the right of condemnation, permission to exist perhaps as a monopoly, in return for which, in view of some assurance of reasonable profit, we limit the return on the value of the property to a fair amount, which, while greater than that received from a Government bond or municipal security, is nevertheless smaller than would be judged reasonable for an industrial concern, where the risks are greater. In brief, the peculiar conditions surrounding any specific corporation must be considered both in fixing the fair value of the property and fair return thereon. Mr. Floy.

J. Martin Schreiber, M. Am. Soc. C. E.—The author has presented a remarkable analysis, and has approached the subject in a liberal manner, which is certainly required for a rate investigation, if a sound conclusion is to be expected. Mr. Schreiber.

The greatest difficulty is in the practical application of the principles which have been set forth. In discussing the paper by Mr. Riggs,* the speaker brought up the question of fair values in relation to present physical values. Also, in a recent investigation of the depreciation of certain elements of physical property, he interviewed leading specialists eminently fitted to give the information desired. Some of these men replied that they would not even attempt to designate the correct life, if changes in the art were to be considered; and, of course, an estimate of the life of property without the consideration of obsolescence would be valueless. To take a practical case: It is reported† that, in the recent valuation of the Elevated Railroads of Chicago, the value, exclusive of roadway and overhead charges, submitted by the Chicago Harbor and Subway Commission, was \$26 354 217; the appraisement by the A. L. Dunn Company was \$40 750 892, and the valuation by George F. Swain, M. Am. Soc. C. E., was \$34 634 396. Now, it does not appear to be fair to the stockholders to be asked to abide by any method showing such variations. It should be admitted that a fair valuation, representing moneys properly invested for property which has been built up by the piecemeal method, with equipment constantly changing on account of the development of the art, along with unreliable book records, with their varying accounting methods, is a very difficult proposition. This is especially true when one must take into consideration the vague item of overhead charges;

* *Transactions*, Am. Soc. C. E., Vol. LXXII, p. 1.

† *Electric Railway Journal*, May 18th, 1912, p. 829.

Mr. Schreiber. thus far, that item has generally been determined by setting apart an arbitrary percentage. For this reason the speaker is of the opinion that a valuation, particularly of old properties, for rate purposes based on earning power, is more reliable than the method which takes the values of actual physical properties as the governing data. Also, the value for rate making should not, necessarily, be ascertained by the same plan as the value for taxes or bonds.

Assuming that a fair valuation is known, another very pertinent practical question is: "What is to be allowed to the stockholder?" The speaker believes that the general tendency is to place too low a limit on proper earnings. One should not expect to raise money to finance projects which carry with them a large element of risk at savings account dividends, even if the utility appears to be reasonably safe. The investor knows full well that there are still such contingencies as strikes, earthquakes, floods, and lean years, and that it is absolutely impossible to anticipate these conditions; besides, if one expects the country to develop, one must also encourage the honest man with the brains and energy.

Mr. Boucher. WILLIAM J. BOUCHER, ASSOC. M. AM. SOC. C. E.—During a part of 1911, the speaker had certain work in connection with an appraisal of the lines of the Illinois Central Railroad in Illinois. The values given to the right of way in Chicago, from Randolph to 12th Streets, presented some interesting phases.

When the road was constructed, about 1853, this section was built on piles and over the waters of Lake Michigan, to a terminal at Randolph Street. This space has since been filled in, the width of the right of way varies from 100 to 400 ft., is depressed some 12 ft. below the general level, and is contained between retaining walls, beyond which, on the east and west sides, lies Grant Park. The railroad occupies a most unique and valuable grant for about 7 miles south from Randolph Street, lying, as it does, along the shore of the Lake, giving space for eight running tracks, a roundhouse and car-cleaning and storage yards. It is a popular notion, which the speaker cannot verify, that the land cost the railroad company nothing, but was granted as an inducement to enter the city.

The westerly edge of the right of way through Grant Park is 500 ft. or more from the property line of Michigan Avenue, the nearest property which is ever for sale and has a definite, taxable value. The Company, in its appraisal, placed a square-foot value (seemingly very high) on all the right of way, in addition to improvement values, such as filling, which the Company had done, beginning at Randolph Street and increasing toward the south until it reached a maximum opposite

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Jackson Boulevard, where property values along Michigan Avenue are the highest, and then decreasing again until 12th Street was reached.

The question then arises: How were the values arrived at for property which had probably never cost the railroad company anything, which is located 500 ft. from the nearest property for a comparison of values, and which to-day could not be acquired for railroad purposes in any manner or under any circumstances? And, further, is a railroad company justified in placing a value in excess of $36 000 000 on such a property for appraisal purposes?

Another matter regarding railroad finances has often interested the speaker: One frequently sees, in the daily press, announcements of the sale of bonds of various railroads, which read somewhat as follows:

"The A. B. & C. R. R. is offering for sale $1 000 000 of 4% bonds, secured by various underlying securities (mentioned). Of this amount, $500 000 is to be applied to renewals of bridges and rails and $500 000 for new cars and new locomotives."

The question that arises is this: Is such use of a bond sale proper, if depreciation on rails, bridges, cars, and locomotives has been charged off? Why not use the funds from such depreciation account? Why is it proper or necessary to issue bonds for these ordinary renewals, which in the usual course of events must be taken care of, and must be expected? It has always seemed to the speaker to be improper. Where is it going to lead the corporation, and where will it end, to keep adding to the capital account and paying interest on worn-out and scrapped material?

Mr. Coombs.

R. D. Coombs, M. Am. Soc. C. E.—In a hypothetical case, such as a corporation having paid for a number of years a very high dividend rate, should those dividends be deducted from the capital before fixing the amount on which they could properly be paid?

To consider an exaggerated case, suppose some corporation had been paying about 50% dividends for four or five years; if the Court decides that the corporation can charge only such a rate as will enable it to pay a fair percentage on present value, must those 50% dividends for four or five years be deducted?

Mr. Van Cleve.

A. H. Van Cleve, M. Am. Soc. C. E.—Mr. Grunsky has undoubtedly rendered a great service to the Engineering Profession in setting forth so clearly the methods to be used in the appraisal of public service property for rate-making purposes. While there are marked differences of opinion as to the methods which should be used in such cases, the author's examples and tabulations illustrate so completely the principles which he advocates that his results and the methods of reasoning which lead up to them are entirely clear. His statement of fundamental principles is especially valuable, but the speaker begs to differ with him in respect to Principle No. 18 as applied in a water-power

Mr. Van Cleve. development the output of which is used for supplying electrical current to the public. That principle is stated as follows:

"Intangible values should be disregarded, in making appraisals for rate-fixing purposes, excepting only when the rate of net return is deliberately fixed at or too near the rate earned by ordinary safe investments, in which case an arbitrary addition to the appraisal, under whatever name, should be made. The interest on this item of the appraisal will be the reward of the owner for management and for any hazard which the business may involve."

In the case of a water-power development, the value of the water right, which is an intangible value, is one of the most important items to be considered in determining the value of the plant, not only for purposes of sale, but for the determination of the rates which may be properly charged by the owners of that plant for the service which they render; and the consideration of this item is essential, regardless of the question of the rate earned by ordinary safe investments. To neglect the value of a water right would work a grave injustice to its owner, for, in many cases, if only the tangible values of the several parts of the plant are considered by the rate-making body, and the returns to the owner are estimated at an ordinary rate of interest thereon, together with due allowance for amortization and all other items which may properly be included, it would result in compelling him to furnish service to the public at a rate far below that for which the same service could be furnished by a plant producing the same power from any source other than water.

It is true that the author, in Principle No. 17, states: "Proper investments for franchises, for water rights, and the like, are always to be included in the appraisal." It is frequently the case, however, that although no investment has been made directly for the acquisition of the water right, the value, nevertheless, exists, and should always be considered in a valuation. The correctness of this principle may perhaps be most clearly demonstrated by an example:

Let it be assumed that A is the owner in fee simple of lands lying between two streams, the water in one of which is at a material elevation above that in the other; that this property has been in his possession for many years, and that the price paid for the land was the fair and reasonable value of it for farm purposes; that A as a riparian owner has the right to withdraw water from the upper stream and discharge it into the lower stream; and that he has complied fully with all the legal requirements necessary for this purpose. A is then the possessor of a water right for which no expenditure has been made by him. Let it be assumed, further, that in a city located within reasonable transmission distance there is a market for the electrical current which can be produced by the water which may be withdrawn from the upper stream adjoining A's property and dis-

charged into the lower stream, and that A determines to grasp the opportunity to make use of a water right which, before the advances in the art of transmission, was practically valueless, owing to the lack of any market in the immediate vicinity of his property. Having obtained the necessary franchise in that city, in due course he enters into contract for the sale of this power to the municipality and to the citizens thereof, and the plant which he has built therefore becomes a public service property. In the course of time a rate-making body is called on to decide as to the fair and just returns which he shall receive on his investment, and the value of the water right becomes—as it will be shown—by far the most important factor in determining those rates. In order to simplify the illustration, let it be assumed that the just returns are to be determined for power at the city limits after the voltage has been reduced for safe distribution, and thus eliminate from consideration the cost of the distributing plant. Mr. Van Cleve.

It is found that the plant is capable of developing at all times 25 000 e.h.p. at the switch-board, and that the original cost and also the replacement value of the tangible values included in the plant are equivalent to \$40 per e.h.p.; that the corresponding tangible values of the transmission line is \$6 per e.h.p., and that of the step-up and step-down transformer station and switch-board apparatus represent an investment of \$14 per e.h.p., or a total of \$60 per e.h.p. of actual investment in tangible property. The load factor is found to be 50%, and 13% the average loss in transmission and transformation. The power available for sale at the city limits, therefore, is 21 750 e.h.p., and the average power sold throughout the year is 10 875 e.h.p., or 43.5% of the total installed capacity.

Assuming the average life of the plant to be 20 years, and that 5% is a fair and reasonable return on the investment, the annual fixed charges and operating expenses per electric horse-power would then be fixed by the body called on to appraise the property for rate-making purposes about as follows, the figures being based on the investment per electric horse-power:

Interest on \$60, at 6%	\$3.60
Amortization, 3.36%	2.02
Taxes and insurance, 1½%	0.90
Maintenance aud repairs	1.00
General expenses	0.70
Operation (wages and supplies)	1.00
Total	\$9.22

This figure is based on the total capacity of the plant, and is equivalent to \$21.20 per electric horse-power of the average power sold, or less than one-third of a cent per kilowatt-hour. In other words, if the

Mr. Van Cleve.

owner of the property were allowed a fair and reasonable return on his actual investment in tangible property, he would be required to furnish power at the city limits at less than one-third of a cent per kilowatt-hour, a result which is manifestly absurd. It is evident that, although his actual investment in tangible property may have been determined properly, nevertheless, that investment does not in any sense represent the real value of his holdings, on which the rate of return should be determined, and that this error is due to the omission from consideration of the value of a water right which cost him nothing.

Furthermore, it would certainly seem to be a poor law which would omit from consideration, in an appraisal for rate-making purposes, a value on which the owner is taxed; and the Courts of New York State, in the case of The People *ex rel.* Niagara Falls H. P. & M. Co. *vs.* Smith,* have held that a riparian right is taxable. To deny an owner a return on a value for which he is taxed, whether that value is tangible or intangible, is a principle which certainly would not hold in law.

The correctness of including the value of a water right in the value of property on which a franchise tax is to be levied has been upheld in the case of The People of the State of New York *ex rel.* *vs.* the New York State Tax Commissioners.† This case was carried to the Court of Appeals, and the contention of the State Board of Tax Commissioners was fully upheld. It may be interesting to note that, in this case, not only was the value of the water right given due consideration as affecting the value of the entire plant of the power company, but it was further held that the value of that water right should properly be apportioned to the several parts of the plant in the same ratio as that which the tangible values of those parts held to the tangible value of the entire plant. The principle of including in the value of a water-power plant the value of the water right, therefore, has not only the approval of common sense but the sanction of the Courts.

Assuming that the value of a water-power is to be considered in the appraisal of a property for rate-making purposes, the question then arises as to the method by which that value shall be determined. If an actual investment has been made for the acquisition of the right, and it is held that the original cost represents a fair and reasonable value thereof, the case is a simple one, and the principle to be applied has been most clearly and ably set forth by Mr. Grunsky; but, on the other hand, if no outlay has been made for the water right, as in the illustration just given, the problem is a complex one, and no doubt there would be a great difference of opinion as to the manner in which the true value should be ascertained.

* 70 App. Div., 543; 175 N. Y., 469.

† At an extraordinary term of the Appellate Division, November 22d, 1909, Hon. L. W. Marcus, Justice, presiding.

The speaker ventures to suggest a method which he has used, and believes determines fairly the procedure to be followed. Reverting to the foregoing illustration, first determine, for the sake of comparison, what annual returns a rate-making body would allow the owner of a modern steam plant for the development of the electrical current for which a sale could be effected in the city under consideration, namely, 10 875 e.h.p., average use. Let it be assumed that the investment per electric horse-power of rated capacity is $60; but, as the total capacity of the plant, for the purpose of comparison with the water-power development, need be only 21 750 e.h.p.—the amount of power which the latter could deliver at the city limits at a suitable voltage—the actual investment per electric horse-power for purposes of comparison should be $52.30. It is understood, of course, that, in determining the rate of return which the owner of a steam plant should receive on his investment, the higher value should be used, but, as the comparison is between a steam and a water-driven plant, the lower value will be used in this illustration. As the life of a steam-driven plant will be less than that of the water-driven plant, it will be assumed, for the sake of comparison, that the life of the former is 15 years. Mr. Van Cleve.

The comparative fixed charges and operating expenses for the steam plant having a capacity equivalent to that of the water-driven plant will be approximately as follows:

Interest on $52.20, at 6%	$3.13
Amortization, 5%	2.61
Taxes and insurance, 1½%	0.78
Maintenance and repairs	1.30
General expenses	0.70
Fuel (coal, $2.25 per gross ton)	13.13
Wages and supplies	2.00
Total	$23.65

The speaker recognizes the fact that there may be a material difference of opinion as to the amount which should properly be charged to the items of maintenance, fuel, and wages, but it is believed that the foregoing figures are conservative in that they do not exaggerate the operating expenses of a steam plant, under the conditions set forth, as compared with those of a water-driven plant. This is shown further by the fact that the total annual fixed charges and operating expenses per electric horse-power are equivalent to only 0.7 cent per kw-hr. of power development, which is certainly a reasonable figure for a plant of the capacity assumed.

Assuming the correctness of the foregoing illustrations, it would appear that the additional annual return which would be allowed by the rate-making body to the owner of a steam plant would be $23.65,

Mr. Van Cleve. while, if the value of the water right be disregarded, the owner of the water-driven plant would be allowed to receive for exactly the same service only \$9.22, an annual difference of \$14.43. The unfairness of such a proposition is self-evident. If the owner of the water-power plant is to be allowed to receive a like return for like services, the value of the water right must be determined by capitalizing the difference between the annual yearly expenditures as above set forth, which is \$14.43. If it has been determined that the reasonable interest on money invested is 6%, a like rate should be used in capitalization. If, however, the value of the water right is to be taxed at 1%, the rate of capitalization to be used should be 7%, on which basis the value of the water right is \$206.14 per e.h.p. of rated capacity, an amount far in excess of the investment per electric horse-power in tangible property. To omit this value from consideration in appraising the property for rate-making purposes would lead to a far greater error than to omit the entire value of the tangible property.

While the speaker is not aware that the Courts have passed definitely on the correctness of the foregoing principle for determining the value of a water right, this method has been brought before the Courts in the case of the Fulton Light, Heat and Power Company *vs.* The State of New York, and in the Franchise Tax case previously referred to. From the award given in the former case, it would appear that the Courts certainly gave grave weight to the above method which was used in determining the value of a water right. In the finding of the trial judge in the case of The People of the State of New York *ex rel. vs.* The New York State Tax Commissioners, and in the decision of the Court of Appeals thereon, no definite approval of the foregoing method for determining the value of the water right was given, but, on the other hand, there was no criticism of it, and it was concluded definitely that the right had a value which should be considered. Owing to the fact that the appellant set forth no theory as to the value of the water right, but stated merely that he had owned the canal for 30 years, and did not know what it was worth, it was shown that there was no necessity for the Courts to pass on the correctness of the theory advanced by the respondents.

While the speaker is aware that opinions will differ as to the methods to be used in determining for rate-making purposes the value of water rights, he has suggested the foregoing in the hope that it may lead to discussion and suggestions, and the advancement of other theories.

There can be no doubt that the question is one which cannot be passed over lightly, and that it will become of increasing importance in the future, as water-power development is proceeding rapidly, and engineers will be called on more and more frequently, in making appraisals for rate-making purposes, to determine the value of the

water right on which the owner of a public service corporation is entitled to returns. The fact that the water right cost the owner nothing, or that his investment for its acquisition was merely a nominal one, should have no real bearing on such determination. Mr. Van Cleve.

The speaker is aware that the illustration which he has used herein may be considered as an extreme one—although supported by fact in actual plants now in operation—but the real question is not as to whether the details are in accord with the opinions of all, but rather as to whether the general principles to which attention has been called are correct.

W. Kiersted, M. Am. Soc. C. E. (by letter).—The painstaking manner in which Mr. Grunsky has treated the question of the appraisal of public service properties, particularly for rate-fixing purposes, is decidedly interesting at a time when the valuation of public utilities occupies so prominent a part of the work of engineers associated with municipal and public service corporations. The views appertaining to matters of valuation in its various aspects are certainly at variance, although perhaps not exactly divergent. Much is being written on the subject, and the efforts of one who has given appraisal as much study as the author appears to have done, in order to outline fundamental principles and proper rules of action, are certainly most welcome, and should receive the hearty encouragement of every one interested therein. Mr. Kiersted.

In 1897, the writer's paper, "The Valuation of Water-Works Property,"* and the discussion thereon, developed a divergency of views which was to be expected at that time. Since 1897 the many calls on engineers to value water-works properties and other public utilities, for rate-fixing, purchase, and other purposes, has so inspired the study of the various problems entering into the valuation of public utilities that to-day sufficient experience and knowledge should have been accumulated to admit of outlining more uniform methods of valuation for any and all purposes than have prevailed heretofore, were the experience in this line of work to be directed methodically, through concerted action, to outlining essential and fundamental principles and general rules of work.

There may be no intentional bias on the part of most engineers, acting individually, in their efforts to formulate fundamental principles and rules affecting, and guiding in, the appraisal of public service properties for rate-fixing and other purposes; but is it not true that the individual sometimes becomes imbued almost involuntarily with the idea that certain methods or theories representing the fruits of much labor and of frequent use are right simply because they are familiar; is it not true that frequently the individual discovers him-

* *Transactions*, Am. Soc. C. E., Vol. XXXVIII (1897), p. 115.

Mr. Kiersted. self attempting to adapt lines of practice to a theory worked out in a sort of academic way, instead of first studying thoroughly the practical side of the questions involved and formulating rules and methods in accordance with well-established and good lines of practice; and is it not true that the assumptions or illustrations often set forth to illustrate a theory are in part or wholly incompatible with the ordinary and permissible lines of practice, particularly in arguments presented in defense of a theory?

It seems to the writer that one of the essential things to be done, in advance of expending efforts to prescribe rules and methods to guide in the valuation of public utilities, is to determine what the fundamental principles governing questions of this kind may be, not necessarily those elaborated in the office, but those determined through an intimate knowledge of all the questions and all the problems entering into the organization, construction, operation, maintenance, and development of any public utility. Until these principles shall have been fully determined and clearly defined, it is difficult to conceive how any practical theory or rules of procedure can be laid down.

In almost every article or discussion relating to the valuation of public utilities, the term "life" is freely used, as though a public utility was constructed for the purpose of serving a specific want only for some definite period of time. This is fundamentally wrong, for every public utility must live as long as there is need for the use of it, whether the period of time be one year, a century, or longer. The error is confusing, and arises from applying to a composite structure a line of reasoning relating solely to the life limitations of various elements or units going to make up that structure, apparently without regard to its wholly indeterminate life. It is known from experience that certain units of a composite property subject to heavy wear and tear become useless in time, others become incapacitated and are no longer able to perform the functions which are expected of them as part of that property, and others are abandoned because progress in the arts and new inventions compel the substitution of new methods and new devices for those in use. In this manner, experience guides in placing a life limit on particular units of a composite property; but the same experience does not sanction the placing of similar limitation on the life of a public utility considered as a whole. The unit may have an approximately determinate life, while the composite structure possesses a wholly indeterminate life.

For instance: a municipality is a complex and composite organization, under State and Federal regulation, made up of individual units, each unit performing some particular function in the body politic; each unit contributes to the support and progress of the municipality; and each unit, while concentrating efforts toward its own business success, is contributing constantly to the progress of the organized

Mr. Kiersted.

unit in which it lives. All the units are blended into one great regulated force, under such regulations and restrictions for mutual support that the elimination, by death, failure, or disappearance, of any unit of the human fabric no more than temporarily affects the progress of the community as a whole. Every municipal public utility is organized and developed for the public need; it thrives upon and is in every way a part of the community. If the conditions which promote the prosperity of the community flag, it may cease to grow, and may even retrograde, creating a business depression which is felt by all the public service corporations, and affects in one way or another even the humblest individual. On the other hand, the prosperity of the one follows in the wake of the prosperity of the other. There is no limit of life to the community and its public utilities, as far as human vision goes, but there is a limit to the life and period of usefulness of the individual units in the community and of the personality of those engaged in operating its public utilities.

In like manner, the various elements going to make up a composite property like a water-works, street-car system, or similar public utility contributing to the welfare, advancement, comfort, and convenience of the community, stand in precisely the same relation to the composite structure as the human element does to the community in which it lives, prospers, and performs some specific function.

It seems to the writer, therefore, that a fundamental error is committed when the term "expectancy," or "life," is applied to any composite structure as it would be applied to an element or unit of that structure which is known from experience to have a more or less fixed and definite period of service. The structure as a whole is an enduring one, as long as there is a demand for it in the community which it serves. To remain serviceable, it must be extended and improved from time to time; its worn-out units must be replaced with new, larger, and better ones; new methods of operation must be substituted for old and antiquated ones; business organizations must improve, in order to meet the demands of advancement in economics; and betterments must be made from time to time, to meet the various rules and regulations imposed by those who have been made the guardians of the public welfare and the public health and comfort. In short, any public utility, to be of service to the community which it is organized and constructed to serve, must be maintained in an efficient, up-to-date condition; and, in order to do this, renewals, replacements, and extensions must be made periodically. It cannot die; and it cannot be allowed to retrograde, as long as the community depending on it lives and prospers and the need for it continues.

Again, the progress of any community is by no means uniform. Certain conditions promote a rapid growth at one period which adverse conditions at a subsequent period may check temporarily. Such periods

Mr. Kiersted. of growth and stagnation arise in the development of any public utility; nearly every decade registers a jump in structural costs arising from numerous replacements and general betterments in the line of progress. A composite property possessing an average age of 15 years may within 2 or 3 years have this average age nearly halved. Very seldom, in a progressive city, does the average age exceed 20 years. An illustration of how quickly the average age of a composite structure may change is that of a water-works property which was purchased in 1909 at an appraised value of $1 100 000, in round numbers, when the average age was 15.5 years. In 1912, after extensions had been made aggregating more than $820 000, the average age of the physical property was 8.7 years. The original construction of this property was started in 1883; extensive changes were made in 1886, again in 1898, and again in 1909. Between the periods of heavy expenditures there were longer periods of a more moderate rate of expansion.

Another case illustrating the irregular growth of a public utility is that of the water-works of Kansas City, Mo. The original works were built by the National Water-Works Company about 1875, with a water-supply connection in the Kaw River; in 1886 new water-supply works were constructed on the banks of the Missouri River, and, at the same time, extensive general improvements were made; in 1895 the city purchased the water-works from the private company, and made some large extensions; in 1905 further extensive and costly improvements were made, followed by other costly improvements in 1911. Aside from these expensive periodical extensions, pipe-line extensions progressed continuously, although somewhat irregularly. In 1912 all the pumping engines purchased in 1895 had been replaced by larger and more expensive machinery; one of the pumping stations had been entirely abandoned, and the other had been modified extensively. The purchase price in 1895 was $3 100 000; the cost of the physical property to-day is $8 000 000, in round numbers; and the average age is approximately 17 years.

Both these illustrations are of public utilities in cities where the growth and development may have been more rapid than in some cities in other localities; but, however this may be, they afford a practical example of how public utilities are developed—the difference in the rate of development, between a rapidly growing and a slowly growing city, being one of degree only. Replacements and rehabilitation periods may be at more frequent intervals in a city of rapid growth than in one of slow development, and the average age of a composite property may be somewhat longer in the latter than in the former, but there can be no such condition in practice as a public utility standing still and progressively growing old. If a community retrogrades, the value of the individual units of city property, of the city property as a whole, and of all its public utilities, must depreciate; and the owner

Mr. Kiersted.

of a public utility must witness the value of his investment grow less and less as the community continues to retrograde.

These two considerations, namely, the indeterminate life of a composite structure like almost any public utility, and the comparatively short average age under almost any conditions to be found in any one of our American cities, are believed to be essential and fundamental, and to underlie all practical methods of appraising public utilities.

The estimated cost of duplicating the physical property of a public service corporation is not in itself a measure of the value of the property; it is simply an essential element of the value of such a property. The more completely replacements and substitutions of perishable and incapacitated units may have been made and the entire plant may have been maintained in a serviceable condition, the nearer does the fairly estimated cost of the reproduction of the physical property represent its full reproduction value as an element of total value of the entire property. It is seldom, however, that this is the situation when an appraisal is to be made, and consideration should be given to the measure or degree in which the physical property falls short of this ideal condition.

The author regards real estate, or rather the increment of value of real estate over and above the purchase price, as reinvested earnings open to consideration in an appraisal for rate-fixing purposes. This view seems to be entirely consistent with that generally entertained at the present day, and with the theory of valuation based on the cost of reproduction of the physical property under conditions as of the date of valuation. It would follow, naturally, from the same line of reasoning, that any of the physical conditions which increase the actual cost of reproducing any unit or any aggregation of units of the physical property, as compared with the cost at the time of construction, may also be considered with equal consistency as reinvested earnings, and, accordingly, should be included in the capital account subject to consideration for rate-fixing purposes. For instance: the cost of laying water pipes under paved streets and in streets where the working force is subject to great interference on account of the presence of underground structures, notwithstanding the fact that the pipes were actually laid in unpaved streets before the existence of many of the present-day underground structures, has usually been regarded as a proper element of value in connection with cases where the valuation has been for purchase and sale, and has been occasionally questioned in connection with rate cases solely. It would seem, however, that if enhancement of value in one particular is to be considered in rate cases, enhancement of value in other directions should be open to equal consideration. Furthermore, if enhancement of value in any particular is legitimate and fully consistent with the theory of valuation on the basis of reproducing the physical property under present-

Mr. Kiersted. day conditions, it would appear that, on the other hand, it is also perfectly proper that all questions of depreciation of the value of the physical property should be given equal weight. If it is fair to allow enhancement of value, it must be equally fair to consider depreciation of value; both are part and parcel of any method of valuation based on the cost of reproduction under the physical conditions existing as of the date of valuation.

The computation of depreciation, as applied to the appraisal of public utilities, has been rendered somewhat complex and rather inconsistent by the infusion of sinking-fund methods. The limited franchise under which many of the public utilities have been constructed and operated may have served to suggest the use of the sinking-fund method in computing depreciation. The application might be proper enough were the property to pass out of existence at the expiration of a limited franchise, and were the rates to be charged for public and private service sufficient to return to the investor the invested capital with interest; but it is seldom or never the case that the physical property expires with the franchise. There have been a very few instances where an investment has been irreparably injured by a city (after refusing either to purchase the property or renew an expired franchise) constructing competitive works and virtually crowding the owner of a public utility out of business. Occasions of this kind are so few as to be scarcely worthy of consideration in a comprehensive view of the subject, and have usually resulted from either an unfortunately drawn franchise ordinance or a bitter contention precipitated between the owner of the public utility and the city.

Sinking-fund computations find well-defined application to financial problems like those of the redemption of municipal, state, and national bonds, and to problems like those involved in life insurance; but the application of such a method to the financial management of any public utility compels a complication of accounts which scarcely any management of a public utility would care to introduce. Its application in this regard is not only cumbersome, but conflicts with the actual conditions of practice. Moreover, it is likely to prove inequitable. It compels the assumption of a life period for each of the numerous units entering into the composition of a property, and the computation of depreciation by well-known sinking-fund methods for some definite portion of an assumed life term for each of the various units.

The assumption of a life period for the units of a composite property and the computation of depreciation of the various units on the sinking-fund basis is equivalent to computations of depreciation on composite property having an equivalent life period. The operation of the sinking fund, as applied to the depreciation of a property having a composite life of 60 years, is shown by Table 13.

TABLE 13.

Mr. Kiersted.

Age of property, in years.	Percentage of total depreciation, with sinking fund invested at 3 per cent.	Average annual rate of depreciation by decades.	
0	0		
10	7	0.7	1st Decade.
20	16.5	0.95	2d "
30	29.2	1.30	3d "
40	46.2	1.7	4th "
50	69.2	2.3	5th "
60	100.0	3.1	6th "

The average of the six decades is 1.67% per year, corresponding with the straight-life method of depreciation for a 60-year life.

The average rate of depreciation as thus computed is irregular; it is moderate in the earlier years and very rapid in the later years of the life of any unit, and, in the long run, is likely to result in an inequitably proportioned depreciation, a progressive increase of rates to meet interest on sinking-fund investments, and any number of incongruities in accounting and in attempting to harmonize practice with theory. The fact is that the sinking-fund method of computing depreciation assumes the plant to grow old continuously, and finally to wear out and pass out of existence, whereas, in reality, the plant usually grows more efficient as it grows older, through necessary and indispensable replacements of its various units, and the property becomes more valuable.

The sinking-fund method really has no place in computations of the rate of depreciation of a composite property. As long as worn out, incapacitated, and obsolete units and methods of operation are replaced progressively with new, improved, and larger units and methods, and the property as a whole is extended and enlarged from time to time, to meet the demands of a growing community, there seems to be little ground for further consideration of depreciation, unless it may be that referred to as deferred maintenance.

By deferred maintenance is meant deterioration other than that which can be taken care of in the ordinary course of events by current expenditures covering ordinary wear and tear and ordinary replacements. For instance: as time goes on, the discharging capacity or serving capacity of the distributing pipes in a system of water-works is reduced materially by the tuberculation of their interior walls, and this necessitates a cleaning process, or reinforcements, or replacements. Such deterioration is usually allowed to progress until inferior service compels the use of comprehensive and extensive cleaning and replacement.

The average annual cost of replacements of four water-works properties, the histories of which are well known to the writer, when dis-

Mr. Kiersted. tributed over the average age period of the respective properties, is 0.6% of the cost of reproduction.

In another instance, a water-works property, which had been started in 1865 and in 1907 was found to have many of the older pipes heavily coated internally, received a special depreciation of 0.33% per annum of the cost of reproducing the physical property, representing deferred maintenance.

The aggregate annual cost, covering replacements and deferred maintenance, as above stated, of 0.93% of the cost of reproduction, represents a rate of depreciation which is under rather than over that which may be expected to approximate the rate of depreciation of these particular properties, for the reason that some of the minor replacements are not accounted for.

It may be stated, further, that the foregoing replacements are of the important parts of water-works, like pumping machinery, buildings, boilers, settling basins, water-supply intakes, and other important units which are subject to replacement or radical changes at long intervals. They do not in any degree embrace the expenditures covering ordinary repairs and current maintenance. These costs, wholly in excess of that above estimated, would by themselves approximate 0.8 to 1% per annum of the cost of reproducing the physical property. In all probability an allowance of 2% on the cost of reproducing the physical property of a water-works would represent the amount of money to be set aside annually for general maintenance; about half of this would be used to cover current maintenance expenditures, the other half could be set aside as a fund to replace the worn-out and incapacitated units, as becomes necessary in the ordinary course of events.

The writer does not presume to offer these figures for general application, although, for the particular properties considered, and for similar ones, they may not be far wrong. They serve the purpose of illustrating what the writer believes is a simple method of computing the depreciation of the physical part of a water-works property, and one which will commend itself to the bookkeeper as well as to the superintendent and other officers operating a public utility of this particular kind. By varying the annual percentage rate in harmony with the average rate of decay of the various elements going to make up the physical property of public utilities of other types, the same method may become equally applicable. Part of the money annually set aside from the earnings for general maintenance may be apportioned to meet the annual costs of current maintenance and repairs; another part may be a fund for use in making replacements, and for any similar work which would serve to perpetuate and insure the highest degree of efficiency and the continued usefulness of the physical property.

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A straight-line method of this kind for computing depreciation avoids all the incongruities and inconsistencies of the sinking-fund method; it is equitable; it is practical, and in accord with the present methods of operating public utilities; and it simplifies bookkeeping, and can perhaps meet general requirements in this direction more nearly than any other.

In estimating the present value of the physical property of any public utility, it would simply be necessary to estimate the average age of the composite property which is being appraised, and deduct, as depreciation from the estimated cost of reproduction, an amount found by applying the annual percentage rate covering replacements and deferred maintenance multiplied by the age of the composite structure. The annual percentage rate should be determined from general experience and from particular knowledge of the property under appraisement.

The monopolistic character of many public utilities eliminates from consideration the question of the influence of competition to such a degree as to render the method of computing depreciation herein proposed more nearly applicable than would be the case were a property subject to the unrestrained influence of sharp competition, as may be the case in many private lines of business.

Passing to the question of the value of a public utility: It is clear that a distinction may exist as to the value for rate-fixing purposes and for purchase and sale, particularly when circumstances are such that franchise value can properly be taken into consideration. Where the power to regulate rates is exercised in accordance with law by a municipality annually, as in California, or by a commission, there can be no franchise value; but it does not follow that such an element of value of a public utility as that ordinarily termed "going value" or "going concern value" can be similarly excluded, for the simple reason that it should not properly be classed as part of the intangible value, particularly when computed irrespective of earnings. Where income—or, more properly speaking, net income—is used as a basis and means of computing "going value," so called, it is open to this criticism, to a degree at least, because it may partake in part of the nature of franchise value, if the prevailing rates are high and the assumed period of time covered in the use of earnings in computing going value is abnormally long. In no event should an appraisal for rate-fixing purposes consider any element of value based on earnings or income. It does not follow, however, that there is no element of value over and above the cost of reproducing the physical property together with usual overhead charges which it is customary to add to such computations, which should be as open to consideration in appraisals for rate-fixing purposes as for purchase and sale.

Mr. Kiersted. In defining what the writer means, he quotes from his testimony in a recent case where a commission jointly estimated the cost of reproducing the physical property of a water-works, allowing all enhancement of value due to existing market and physical conditions, and deducting for depreciation. The element of value referred to in that line of testimony was defined as "going concern value."

"The method of computing 'going concern value' assumes that the knowledge of the general public with regard to the water service is fully developed, it assumes that the business of the existing plant, its patronage, etc., remains intact until delivered to the new plant. No longer time is needed for the recovery of business than is necessary to render the new property administratively and mechanically fit and serviceable for conducting the entire business of the old property, allowing proper time and capital for making the service connections, the getting of machinery into proper working order, elimination of all defects of construction, the duplication of the office records, and the perfection of the business and mechanical organization."

Nearly all the elements of value entering into the computation of the "going concern value," as above described, represent tangible property in one form or another, and may very properly embrace an additional element of operating capital. As thus described, the going value has no relation to the earnings of the property, the reproduction cost of which is being estimated. It may be computed as a percentage of the cost of reproducing the physical property, or in any similar manner.

As thus defined, the going value loses all connection and relation with so-called intangible value; in fact, many of the units entering into or going to make up this going concern value are susceptible to decay, as are other portions of the physical property, and, in rate-fixing cases, should have a percentage of earnings set aside to cover general maintenance as well as interest on the money thus invested.

In appraisals for purchase and sale, there is an additional element of value which may be properly considered, in order to compensate a successful and efficient management. Just what this increment of value should be can only be determined in specific instances, in the light of all the facts relating to the business organization and the progressiveness of the municipality furnishing the patronage for the public utility under appraisement. While this increment of value would not be considered in rate-fixing cases as part of the capital investment, the rates could be adjusted so as to support through surplus earnings an increment of value sufficient to invite investments in a public utility of the kind under appraisement, and to encourage intelligent and efficient management. A community which has had the benefit of such a management can well afford to pay something more, for a property which is well organized and well managed, than it would be willing to pay for one where the situation is reversed.

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On the other hand, a mismanaged property, where earnings have been sufficient to pay a fair return on the invested capital and maintain the property in an efficient condition, but have been diverted from the natural and proper channels, should expect and should receive nothing short of heavy depreciation of value in an appraisal of the property, either for rate-fixing purposes or for purchase and sale. Such considerations as these, entering into the investigations leading to a final conclusion on the value of a public utility, as they properly should enter, involve the intelligent judgment of the appraiser, and force him out of the "straight-jacket" of reasoning through mathematical formulas or any set of rules based thereon.

It does not always follow, however, that the value, either for rate-fixing purposes or for purchase and sale, will be invariably equitable when computed on the basis of the cost of reproduction, for the simple reason that, in exceptional cases, it may give a value in excess of that at which a community can either afford to purchase, or support through its patronage. One instance of this kind in the writer's experience is of a town which retrograded, losing perhaps 40% of its population during the period in which the public utility had been in operation. In this instance there could be no question but what the public utility corporation would have to share in the shrinkage of values resulting from reduced patronage and the inability of the town to meet obligations which otherwise they could have met had the town continued to increase in population and taxable value at the same average rate as it did during its prosperous career.

In view of the many uncertainties surrounding the appraisal of public utilities, it would seem to be well-nigh impossible to formulate any set of rules to guide appraisers in estimating value except those which are of very general application and embrace the more fundamental principles relating to and governing work of this character.

The writer is not one who believes the Courts have led engineers into error in performing their part of the work relating to the appraisement of public utilities. The whole province of the engineer in a matter of this kind is to assist the Court by his evidence, and when engineers of wide experience, but representing opposing interests, present views widely at variance on matters regarding which they are supposed to have an intimate knowledge, the Court is certainly to be excused if the evidence is practically set aside or only partly considered. As a rule, the estimates of engineers of experience on the cost of reproducing a physical property should not and will not vary materially. In fact, a joint conference of engineers representing opposing parties will frequently result in a joint agreement as to values, except possibly on the question of the overhead charges which it is customary to allow in the appraisement of public utilities, and on questions involving going value—elements of value which can be

Mr. Kiersted.

settled quite as readily by the Court itself after hearing individual testimony. Joint agreement on most or all of the technical details, in advance of the hearing of evidence and the filing of a joint report as the evidence of all the engineers, would greatly abbreviate the work of all concerned, and perhaps simplify and materially assist the work of the Court itself, but, unfortunately, in most instances, counsel of the opposing parties do not sanction the joint conference.

One word more, in order to make more clear the writer's position with regard to capital investment: The earnings covering depreciation, in the sense in which the writer has used depreciation, are expected, among other things, to return to the investor the cost of abandoned property. Accordingly, as property is abandoned it should be charged off the capital investment account, and, in a similar manner, any unit substituted for an abandoned unit should be added to capital investment account. Thus the accounts would show the total investment as well as the active or present investment at any date of valuation.

Whatever of criticism there may be in these remarks attaches to the writer with perhaps even more force than to Mr. Grunsky, for the writer has used the sinking-fund method in computing depreciation in numerous instances, but believes that in it he has observed fallacies which should be eliminated, even though it involves the discarding of sinking-fund methods and the radical modification of past lines of procedure. He has nothing but thanks and congratulations to offer the author for presenting in such a comprehensive manner the results of his labors, and hopes that the issues which are thereby raised will be discussed fully and, finally, will result in simplifying and improving the methods used in the valuation of public service properties.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

AIR RESISTANCES TO TRAINS IN TUBE TUNNELS.

Discussion.*

BY MESSRS. GEORGE GIBBS, GEORGE H. PEGRAM, CHARLES S. CHURCHILL, AND L. J. LE CONTE.

Mr. Gibbs.

GEORGE GIBBS, M. AM. SOC. C. E.—In this interesting paper Mr. Davies refers to certain experiments which the speaker conducted in the Pennsylvania Railroad Company's New York Tunnels; these tests, however, were not to determine train resistance, as Mr. Davies apparently infers. While it has been the speaker's hope that he would be able shortly to make tests for this purpose, it has been impossible, thus far, to arrange for them. His experience in conducting such tests at various times during the past twenty years and, it may be said, under conditions more favorable for the purpose than those obtaining in the river tunnels in question, leads him to hesitate to undertake them without somewhat elaborate preparation.

Mr. Davies refers to the difficulties of eliminating from the observations the effect of variable conditions due to acceleration or retardation, changing each moment, almost, during a run, especially on a line involving frequent changes in grade. These have been experienced by the speaker also in interpreting the results of any tests of the kind. Even when using a dynamometer car, containing the most accurate instruments obtainable, it is difficult to eliminate from the records the effects of stored energy, which appear incessantly, continually accumulating or being given out between the cars of a train due to local acceleration or retardation in portions thereof from changes in track or motive-power conditions. In the case of tube tunnels, where the grades are not uniform, in fact where the profile is one long vertical

* This discussion (of the paper by J. V. Davies, M. Am. Soc. C. E., published in *Proceedings* for April, 1912, and presented at the meeting of May 15th, 1912), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Gibbs. curve, and where the run is relatively short, there would seem to be special difficulty in eliminating the variables above referred to; and, moreover, another variable is doubtless produced by currents, or eddies, in the moving air ahead of or following the train. The speaker's experiments indicate that eddies occur, even from very slight local obstructions along the tunnel walls, such as signals, etc., and at openings of cable manholes, ventilating ducts, and shafts. These eddies, it would seem, must have considerable influence on the air friction ahead of and at the side of the train, inasmuch as they are quite observable on a barometer placed in the car.

The tests in the Pennsylvania Tunnels were primarily for a determination of the ventilating conditions, and secondarily for an analysis of certain very sudden fluctuations of pressure which at high speed produce an effect in the ears of passengers in the trains.

The speaker's paper on the "New York Tunnel Extension of the Pennsylvania Railroad"* describes the emergency ventilating system installed in the tunnels. This system consists of a series of pressure fans arranged with the Churchill form of nozzles to blow air into the tunnels in the direction of traffic and thus induce a sufficiently rapid movement of the air volume for satisfactory ventilation in the case of stalled trains. It was believed that the normal movement of trains which are scheduled at a high speed would produce sufficient piston action to give satisfactory ventilation, and it was the purpose of the tests to determine the facts accurately.

The Pennsylvania Tunnels are lined throughout with concrete, thus giving quite a smooth interior surface. The cross-sectional area of the river tube tunnels is 224 sq. ft., as compared with 166 sq. ft. in the Hudson and Manhattan Railroad Tunnels; but as the cars are larger than those of the latter railroad, the ratio of the cross-section of the car to that of the tunnel is 0.505, as compared with 0.54 for the Hudson and Manhattan Railroad. These conditions, therefore, in the two tunnels are nearly the same, except in portions of the Hudson Tunnels which are not concrete lined, where the ratio of car to tunnel is slightly greater. An important difference in the latter case is introduced by the presence of cast-iron segment flanges, which, according to the speaker's observation, are likely to cause a considerable amount of friction in the moving column of air.

Table 2 gives the maximum and average velocity of the air column caused by the passage of suburban trains at various speeds through the Pennsylvania Tunnels and shows that the air column attains a speed of from three-fourths to two-thirds of that of the train, depending on the speed of the latter; and the average speed of the air column for the entire time the train is in the tunnel is about one-half of the maximum and about one-third of that of the train.

* *Transactions*, Am. Soc. C. E., Vol. LXIX, p. 226.

TABLE 2.

Mr. Gibbs.

Speed of train, in miles per hour.	Maximum observed air velocity, in miles per hour.	Average observed air velocity, in miles per hour.
20	15	7
30	24	12
40	30.5	15.5
50	36	18.5
60	39.5	21.0
70	43.5	23.5

The tests indicate that a suburban train moving at normal speed will push ahead of it, during its run from the portal to the station, about 1 000 000 cu. ft. of air, out of a total of 2 800 000 cu. ft. in the tunnel. After the train has left the tunnel it is found that the current of air in the tunnel remains in motion for about 5 min., but at constantly decreasing velocity, from which it is estimated that a suburban train of average length will replace about one-half of the air content of the tunnel by fresh air drawn in from the portal and from the shafts. Closing the intermediate shafts was found to reduce the ventilation somewhat, but not greatly.

Satisfactory ventilation in a car requires about 30 cu. ft. of fresh air per minute per passenger, calling for 50 cu. ft. of air to be renewed outside of the car; which means that during the rush-hour traffic a complete renewal of the air in the tunnel is required every 20 min. If, therefore, the movement of one train renews one-half the air in the tunnel, the passage of a train every 10 min. will produce satisfactory ventilation. Rush-hour service, however, means a train spacing of about $2\frac{1}{2}$ min. apart; it follows, therefore, that during this worst period four times the required ventilation is produced by the train movements, without the help of the ventilating fans.

The fluctuation of air pressure inside the cars of a suburban train was observed by placing a sensitive barometer in the motorman's compartment at the head, and the air pressure in the tunnel itself was obtained by similar barometers placed on the bench-walls near the entrance and at the shafts. The curve, in the diagram, Fig. 8, shows the maximum sudden fluctuation or change in air pressure in the car as it passes the tunnel portal and at the open shafts, at various speeds. These pressures build up quite suddenly, and fall again as rapidly, after passing the points mentioned, nearly to normal and only slightly above the atmospheric pressure outside of the tunnel. It may be thought that this latter result is inconsistent with the theory that there is a constant pressure ahead of the train due to the fact that the air column is not moving at the same speed as the train, and to the fact that pressure must be produced in overcoming friction. This indicates the great difficulty encountered in obtaining consistent

Mr. Gibbs. results from such tests; in fact, the net pressure in the cars seems to be the resultant of that produced by the eddying currents around, and the suction at the sides of, the cars, as well as by inertia and friction. Therefore, all that can be decided definitely from the tests is that a violent surge is produced as the train enters the portal, and as it passes the shafts, and that the effect at the open shafts is nearly as great as that at the portal. Closing the shaft openings reduces the surge, but does not eliminate it entirely, in fact, it is found that even the small

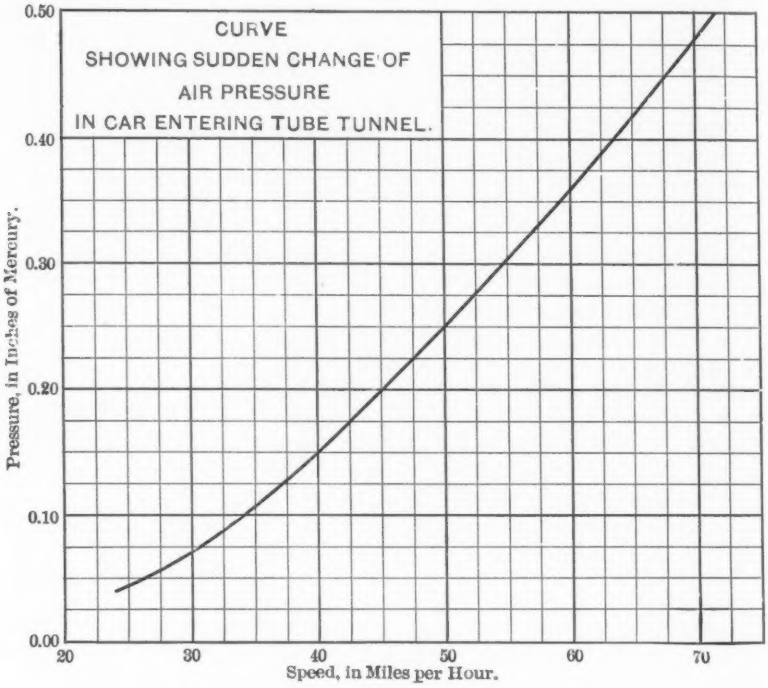

FIG. 8.

air duct and stairway openings produce local disturbances, and these cannot be entirely eliminated as long as there is any variation in the cross-section of the tunnel.

It is also a curious fact that fluctuation of pressure inside the cars is noticeable, although somewhat reduced in amount, when all the windows and ventilators are shut. This shows that a very small addition to the air volume in the car causes the effect—that it is a "barometric" and not a wind pressure which is felt in the ear. Probably, also, the slight deflection of the car sides from an outside

variation in pressure contributes to the result. At slow speeds, say, up to 30 miles an hour, there is hardly any noticeable effect in the ears of persons in the train, and the disturbance becomes disagreeable only when speeds of 50 or 60 miles an hour are attained, and the effect is only momentary. Mr. Gibbs.

The speaker is indebted to the Test Department of the Pennsylvania Railroad for many of the test results used as the basis of this discussion.

GEORGE H. PEGRAM, M. AM. SOC. C. E. (by letter).—There has not been time, since this paper came to the writer's notice, to make the necessary tests for such a discussion as the subject merits. It is a valuable and timely addition to our knowledge of tunnels. The magnitude and difficulty of the task of deducing formulas for air resistance, which will serve as guides in the design of tunnels, become very apparent when tests are attempted. Mr. Pegram.

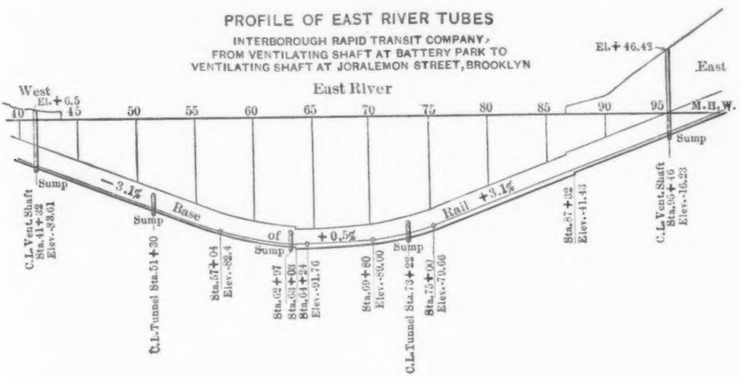

FIG. 9.

The writer has made several tests in the East River tubes of the Interborough Rapid Transit Company between the Battery shaft in Manhattan and the Willow Place shaft in Brooklyn. When tabulated, however, they contain irregularities which cannot be explained, and they are presented in the light of a study to be interpreted in connection with the manner of making the tests and should not be accepted as final results. These tests are shown in Table 3. Fig. 9 is a profile of the tunnel.

The tests indicate that Mr. Davies' Formula 22, page 380,* will probably give good results for total pressures in tunnels of this cross-section, but they show nothing as to the factor, L, which is an important part of this formula. L is the distance between the train

* *Proceedings*, Am. Soc. C. E., for April, 1912.

Mr. Pegram. and the air outlet. It would seem that this factor should be connected with the total length between the air outlets, because the column of air following the train has an effect similar to that preceding the train.

The Battery Tunnel is 5 400 ft. long, between the ventilating shafts in New York and Brooklyn, and has a cross-section nearly identical with that of the Hudson and Manhattan tube shown by Fig. 2, except that a lining of concrete fills the spaces between the flanges of the iron, giving a smooth bore. The clear sectional area is 158 sq. ft., as compared with 160 sq. ft. in the Hudson tube. For 2 400 ft. of this length, however, a reinforced concrete lining is used, which contracts the area in this portion to 145 sq. ft. The section of the cars is 90 sq. ft., the same as those of the Hudson and Manhattan. The ventilating shafts at the ends are 14 ft. wide in the length of the tunnel, and 41 ft. long, with a diaphragm in the middle separating the tubes.

There are openings between the tubes at the three sumps, but these are kept closed with doors flush with the tunnel walls. The ventilating shafts provide free outlet of air to the surface. The conditions would seem to present a good opportunity for getting valuable results.

The tests were made during the regular operation of the tunnel, between 1 P. M. and 4 P. M. on May 10th, 13th, and 14th, 1912. Three observers were located at a point on the duct bench at the side of the tube. One faced the approaching train, reading an aneroid barometer held steadily in the hand, another recorded the readings, and a third, with a stop-watch from which the speed was calculated, took the time required for the train to pass. Frequent readings were made, but those used in Table 3 were the passing of the head and rear of the train, respectively. Readings of the barometer were also taken at the shaft and at Bowling Green Station.

A few tests were made with the aneroid barometer on the front and rear of the train, with the instrument in the position used in the Hudson and Manhattan tests. The speed in these cases was taken by an observer with a stop-watch on the tunnel bench at the stations noted. These results are shown in Table 3.

The most discouraging feature of the observations was the fact that, during the approach of a train, sometimes at a distance of a quarter of a mile, the instrument would show a pressure twice as great as observed as the head of the train passed the barometer, and this pressure would rise to three times the head pressure at a distance of somewhat more than a train length; also, an observer standing in the ventilating shaft at one side of the track would observe a normal pressure, while, in the middle of the track facing an approaching train, he would note an added pressure of about 5 lb. per sq. ft., doubt-

Mr. Pegram.

TABLE 3.—AIR RESISTANCE TEST. EAST RIVER TUBES OF INTERBOROUGH RAPID TRANSIT COMPANY.

Between Ventilating Shafts at Battery Park, Manhattan, and Willow Place, Brooklyn.

Battery Park Shaft, Station 41 + 44, Elevation — 33.68′

Willow Place Shaft, " 95 + 46, " — 16.23′

Clear Sectional Area of Tubes, Station 41 + 44 to 63 + 24 = 158 Sq. Ft.

" " " " " " 63 + 24 to 87 + 32 = 145 " "

" " " " " " 87 + 32 to 95 + 46 = 158 " "

Sectional Area of Car = 90 Sq. Ft.

North Tube to Manhattan.

Station.	Number of cars.	Speed of train, in miles per hour.	Pressure, in Pounds per Square Foot. Front of train.	Pressure, in Pounds per Square Foot. Rear of train.	Pressure, in Pounds per Square Foot. Total.	Total resistance to train.	
52 + 00	8	29.8	4.7	2.1	6.8	612	Observations taken on bench.
52 + 00	8	27.0	5.4	2.1	7.5	675	
58 + 60	8	34.2	2.3	6.0	8.3	747	
63 + 50	8	40.0	3.9	6.4	10.3	927	
63 + 50	8	36.0	3.8	7.5	11.3	1 017	
66 + 25	8	36.0	5.3	8.2	13.5	1 215	
66 + 25	8	36.0	5.3	8.6	13.9	1 251	
66 + 25	8	31.2	4.5	10.9	15.4	1 386	
70 + 00	8	48.4	5.6	13.9	19.5	1 755	
70 + 00	8	39.0	7.1	2.3	9.4	846	
70 + 00	8	36.9	6.0	7.5	13.5	1 215	
79 + 75	8	43.8	12.7	11.3	24.0	2 160	
79 + 75	8	43.8	8.3	5.3	13.6	1 224	
79 + 75	8	35.0	8.3	6.5	14.8	1 332	
79 + 75	8	31.9	8.3	6.8	15.1	1 359	
90 + 00	8	38.9	9.0	4.5	13.5	1 215	
90 + 00	8	31.9	5.3	8.2	13.5	1 215	
94 + 50	10	39.7	10.5	1.2	11.7	1 053	
94 + 50	10	32.0	10.5	1.9	12.4	1 116	
94 + 50	10	21.0	3.0	0.8	3.8	342	
79 + 75	8	37.0	7.9	16.0	23.9	2 151	Observations taken on train.

South Tube to Brooklyn.

	Station.	Number of cars.	Speed of train, in miles per hour.	Pressure, in Pounds per Square Foot. Front of train.	Pressure, in Pounds per Square Foot. Rear of train.	Pressure, in Pounds per Square Foot. Total.	Total resistance to train.
Observations taken on bench.	46 + 00	10	30.6	6.0	4.9	10.9	981
	47 + 00	10	33.6	12.8	5.2	18.0	1 620
	55 + 00	10	34.9	14.3	4.9	19.2	1 728
	57 + 00	10	38.8	10.5	9.0	19.5	1 755
	60 + 00	10	42.6	12.8	10.5	23.3	2 097
	60 + 00	10	42.6	11.3	9.0	20.3	1 827
	60 + 00	10	41.6	10.5	11.3	21.8	1 962
	60 + 00	10	35.6	8.6	6.4	15.0	1 350
	60 + 00	10	32.3	9.8	5.6	15.4	1 386
	63 + 00	10	44.8	9.8	8.3	18.1	1 629
	63 + 00	10	38.8	6.8	9.8	16.6	1 494
	63 + 00	10	34.9	4.5	6.4	10.9	981
	68 + 00	10	29.6	7.9	3.0	10.9	981
	71 + 75	10	38.8	8.3	5.3	13.6	1 224
	71 + 75	10	34.9	6.0	6.0	12.0	1 080
	71 + 75	10	34.9	2.3	6.0	8.3	747
	71 + 75	10	32.3	6.8	5.6	12.4	1 116
	71 + 75	10	31.7	2.3	5.6	7.9	701
	71 + 75	10	29.6	6.0	4.5	10.5	945
	91 + 10	10	25.7	3.8	10.5	14.3	1 287
Observations taken on train.	60 + 00	8	42.0	9.4	14.6	24.0	2 160
	60 + 00	8	34.0	5.0	12.7	17.7	1 593

Mr. Pegram. less caused by the momentum of the air passing along the line of the tunnel.

It would seem as if the relative areas of the tube and the car should be introduced into the formula to make it generally applicable.

It is evident, further, that tests must be made in tunnels of different sizes and with the various conditions of outlets, in order to deduce a general formula, and it is incumbent upon engineers, generally, to supply such data as they can.

The proposed wind shield in front of the train will probably have very little effect in a tunnel for want of a surrounding space into which to deflect the air.

The Battery tubes are provided with blowers at the ends, and it is possible that their efficiency in reducing air resistance might in a measure be determined, although the blowers are installed simply for the purpose of blowing back smoke from either end in case of fire, to allow safer exit to passengers.

Mr. Churchill. Charles S. Churchill, M. Am. Soc. C. E.—Although this subject is very interesting, and Mr. Davies has the thanks of the Profession for having undertaken these tests, it is extremely important that conclusions too far-reaching be not drawn at this time. The author states that this tunnel is especially small, relative to the cars. Although it is not large, there are several steam railroad tunnels which are no larger. For example, Elkhorn Tunnel, on the Norfolk and Western Railway, has practically the same ratio; and in ventilating that tunnel the method of driving the air ahead of the train was adopted. One can sit in a car at the rear of a train and fresh air is felt coming into it through the operation of the fans.

In other cases—at the Washington Tunnel, and also at Big Bend Tunnel, on the Chesapeake and Ohio Railway—the air is forced against the train. In the Washington Tunnel no smoke comes out at the station end, and yet the train drives a certain amount into the large tunnel area which is behind the station, as the train comes out into that area. All the smoke, however, is forced out finally at the other end of the tunnel.

These general points are mentioned in order to make clear the fact that the resistance decreases rapidly after the air is put in motion. The very principle of ventilation that has been used in these tunnels takes this into account; and it is a fact that when air is forced into a tunnel at one end, all of it goes out at the other, and there is no large cumulative resistance.

Now, it does not make any difference whether the tunnel is 1 000 or 5 000 ft. long, the resistance is not very great. Of course, it varies with the length, but it is questionable whether the pressure curve takes the extreme shape shown in the upper curve of Fig. 4.

The City and South London Railway Tunnel, under the Thames River and beyond it, has a section 1.4 times the train section. One section of the Hudson and Manhattan Tunnel is 1.6 times that of the train. Observations in the City and South London Tunnel in 1906 showed that very little ventilation was secured by the train, and about 17 000 cu. ft. only were moved for a period of 1 min. upon the passing of the train, only about 7 000 cu. ft. within ½ min. after the passage of the train, and only a small quantity moved through the shafts.* Mr. Churchill.

These points are mentioned, not for the purpose of giving any exact data, but simply because they have some bearing on any conclusion that may be drawn from this investigation.

The tests by Mr. Davies have been started in directions which should lead to some very valuable results, but it is undesirable to draw conclusions too quickly, and it is important that further tests be made in order to develop this subject.

L. J. Le Conte, M. Am. Soc. C. E. (by letter).—This paper is an extremely valuable contribution to the records of the Society. All the useful facts are conveniently concentrated on the diagrams, and the results speak volumes of reliable information, the full value of which cannot now be over-estimated. Mr. Le Conte.

It has occurred to the writer that another comparison might be of some interest. As Fig. 6 gives resistance curves for 2 000, 4 000, and 6 000-ft. iron tunnels, an interpolation between the 2 000 and 4 000-ft. curves would give a curve for a 3 000-ft. tunnel, and the results might be compared with those for a 3 000-ft. concrete tunnel. This has been done, and the results are quite remarkable, as shown in Table 4.

TABLE 4.—Comparison of Resistances in 3 000-Ft. Iron and Concrete Tunnels.

Speed, in miles per hour.	3 000-ft. Tube Tunnel.	3 000-ft. Concrete Tunnel.	Difference.
15	255 lb.	180 lb.	75 lb.
20	438 "	280 "	158 "
25	643 "	420 "	223 "
30	920 "	575 "	345 "
32	1 200 "	750 "	450 "

The author, however, states that the air outlets are not the same in the two cases, and, therefore, a strict comparison cannot be fairly made. Nevertheless, the results are very interesting as showing, in a general way, the great effect of the interior roughness of tunnel linings.

* *Transactions*, Am. Soc. C. E., Vol. LVII, p. 227.

Mr. Le Conte.

The author speaks of the net end area of the car not being the effective area producing resistance. This same difficulty permeates the entire science of hydraulics and pneumatics. It arises, of course, from the natural viscosity of water and air, particularly at high velocities. The effective area of the resisting object increases with the velocity, hence the resistance increases faster than the simple square of the velocity, as is usually assumed. Of course, the wind shield certainly would play a very important part in reducing this effective area, especially if it were built on true parabolic lines. If it is properly built to suit the highest speed of the trains, the saving in resistance will be fully 30%—that is to say, when the free space is 44% of the tunnel section.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

THE LARAMIE-POUDRE TUNNEL.

Discussion.*

By Messrs. C. Raymond Hulsart and Burgis G. Coy.†

C. Raymond Hulsart, Assoc. M. Am. Soc. C. E.—The maximum length of tunnel driven in one month on the new Catskill Aqueduct for New York City was 523 ft., in September, 1910. This was on the Wallkill Tunnel, a siphon or pressure tunnel, 4.4 miles long, in Hudson River shale. The foregoing record was made in the north heading of Shaft 3, one of six shafts giving access to the tunnel. Mr. Hulsart.

The tunnel cross-section is circular with a required average diameter of 17 ft. As actually excavated, the average diameter is nearly 18 ft. The circular section was found to be a difficult one to excavate in the lower half without trimming. The upper half was excavated closely to line by the top heading, but the bench left tight sides in the lower half which were trimmed after the excavation for the heading and bench was completed. The cross-sectional area excavated by the heading and bench was 230 sq. ft., or 8½ cu. yd. per lin. ft., as compared with the 2½ cu. yd. in the Laramie-Poudre Tunnel.

The Hudson River shale formation consists of layers of sandstone and shale in well-defined beds from a few inches to a foot or more in thickness. The strike is northeast across the line of the tunnel at an angle of 60°, and the dip is southeast about 25 degrees. This rock

* Continued from May, 1912, *Proceedings*.

† Author's closure.

Mr. Hulsart. was fairly easy drilling, a $3\frac{1}{4}$-in. Ingersoll drill cutting about 10 ft. per hour, including changes of steel. There was little trouble in shooting the ground, the cut usually pulling with one shot. The explosive was 60% Forcite, the average quantity used being $3\frac{1}{2}$ lb. per cu. yd. of heading and bench.

Shaft 3, the permanent drainage or unwatering shaft of the siphon, is about 340 ft. deep to the tunnel grade. It is offset 75 ft. from the tunnel and connected with it by a cross-drift. The hoisting equipment consisted of two balanced cages operated by a 150-h.p. Lambert electric hoist. The muck was moved from the heading in $1\frac{1}{2}$-cu. yd. Koppel side-dumping tunnel cars, which were run on the cages and out to the dump. The average haul from the heading to the shaft for the month when the record was made was 2 300 ft. The loaded cars were braked down a 2% grade from the heading and hauled back by mules. A single track, of 30-in. gauge, was used, with the necessary turn-outs and an extra spur at the bench.

The general method of excavation was that of top heading and short bench. The quantities removed were about 5 cu. yd. per lin. ft. of heading and about $3\frac{1}{2}$ cu. yd. per lin. ft. of bench. The drilling equipment consisted of four $3\frac{1}{4}$-in. Ingersoll drills, two mounted on columns and arms in the heading and two mounted on tripods on the bench. Compressed air was supplied from a central plant about a mile away; the air was compressed to 100 lb. at the plant and was supplied at about 85 lb. at the heading.

The tunnel force consisted of two drilling and three mucking shifts of 8 hours each. Each drilling shift drilled and shot a complete round of heading and bench, making two advances a day, which averaged 8.7 ft. or 17.4 ft. per day. The four hours between drilling shifts were utilized in scaling down and mucking back the heading and in setting up columns and drills. Two extra drillers and their helpers, assisted by a part of the mucking gang, did this work. They were usually able to have all the drills set up and one hole drilled when the regular drilling shift came to work.

The heading round consisted of 26 holes, as follows: six cut holes, 12 ft. deep, three on each side about 7 ft. between collars; six relief holes, 10 ft. deep, three on each side of the cut with two breaking-down holes above the cut; and an outside round of twelve 10-ft. holes. The heading was shot in three blasts: first, the cut of six holes; second, the relief and breaking-down holes; and third, the outside round of twelve holes. All the holes of each blast were shot simultaneously, with Victor electric fuses, from the 220-volt lighting circuit.

Each mucking shift consisted of one foreman and twenty-five muckers. This force put up runways, mucked the heading, wheeled the muck in barrows to cars at the face of the bench muck pile, about 100 ft., and shoveled the bench muck directly into the cars. Nearly

150 cu. yd. of solid rock, or 250 cu. yd. of muck, were handled per day. This made 2 cu. yd. of solid rock or 3.3 cu. yd. of muck per man (wheelers and shovelers). Mr. Hulsart.

This tunnel was driven for the Board of Water Supply of New York City by the Degnon Contracting Company. The large amount of muck handled and drilling done during this work was made possible only by the very fine organization and equipment of the contractor.

Burgis G. Coy, Assoc. M. Am. Soc. C. E. (by letter).—The discussions on the Laramie-Poudre Tunnel have brought out many points of interest to the writer, especially when comparing the methods of construction and rates of progress with those of tunnels in other parts of the world. Mr. Coy.

It seems to be the general conclusion that the shorter rounds give the best results and more economical work. This fact was also proven in the Laramie-Poudre Tunnel; the 7-ft. rounds proved more economical and gave greater progress than the 10-ft. rounds, unless the ground was exceptionally favorable, in spite of the fact that the number of set-ups was increased by about 40 per cent. The 7-ft. rounds also made smoother tunnel walls than the 10-ft. rounds, which would mean a saving of concrete if the tunnel were to be lined, and a lower coefficient of friction if left unlined. The direction of the tunnel was nearly parallel with the formation of the rock, which undoubtedly made it more difficult, both to drill and to shoot, than if it had been cross-cutting the formation.

Except where the ground was very soft, the round usually consisted of 21 holes, while in some cases, in extremely hard rock, this number was increased. Experiments proved that fewer holes did not break as well, often requiring a second shooting, and as the 18 holes drilled from the upper set-up could be drilled by the time the muck was out, very little if any time would be saved by drilling fewer holes. It was also found that the greater the number of holes, the smaller the muck was broken, and therefore the more easily and quickly removed. Very few holes failed to break bottom with one firing.

The writer's experience has been mostly with Leyner drills; therefore a comparison of the merits of Leyner drills with those of other makes will not be attempted, but some of the favorable points of this drill are its light weight, making it easily handled, and quick to set up even in close places, the absence of dust at the tunnel face even when drilling "up" holes, and the ease with which the drills can be changed. The holes are easily kept straight, even in broken and seamy ground, and there was almost no difficulty with stuck drills, even when using the 10- and 12-ft. steel.

Three men each made a monthly average of more than 60 lin. ft. of hole per 8-hour shift, the best averaging 61.86 ft.

Mr. Coy.

The time required for the various operations was as follows:

Exhausting smoke from face. . 10	to	12 min.
Picking down roof and sides. . 5	to	10 min.
Jacking cross-bar in place. . . . 6	to	8 min.
Attaching drills, making hose and water connections. . . . 5	to	15 min.
Drilling from top set-up. . . . . . 3 hr.	to 4 hr.	15 min.
Dropping horizontal bar to lower position. 15	to	20 min.
Drilling on lower set-up. . . . . . 1 hr.	to 1 hr.	15 min.
Removing drills, cross-bar, hose, etc. 15	to	20 min.
Blowing out holes, loading, and firing 20	to	25 min.
Ignition to explosion of last hole 8	to	8 min.

Total time required to complete cycle of operation is. . . . . 5 hr. 24 min. to 7 hr. 28 min.

The following cost per foot is furnished by Mr. McIlwee, the contractor:

LABOR:	
Superintendent and foremen	$1.50
Drilling	4.47
Mucking, loading	4.92
Tramming and dumping	4.63
Track and pipe	0.47
Power-house	0.35
Blacksmithing	0.84
Repairs	0.47
Bonus to workmen	1.75
SUPPLIES, ETC.:	
Maintenance of buildings, camps and fuel	0.62
Machinery repairs	0.12
Air drills and parts	1.33
Picks, shovels, and steel	0.84
Explosions	4.50
Lamps and candles	0.42
Oil and waste	0.38
Blacksmith's supplies	0.53
Liability insurance	0.81
Office supplies, telephone, and bookkeeping	0.86
Total	$29.81

As the contractor was to receive a bonus of $300 per day for completing the tunnel before a specified date, some economy was sacrificed to speed of driving, and there is no doubt that the cost could have been lowered somewhat if the bonus proposition had not been considered. The contractor's bid was $32.50 per ft. and the bonus amounted to $63 000. Mr. Coy.

The total overhead charges, including power-plant, camp-buildings and furnishings, pipes, rails, etc., furnished by the company was approximately $120 000. Owing to the location of the work, the value of the plant was very small after the completion of the tunnel, as freight on pipes, rails, building material, etc., from the site to any market would cost more than their value. The entire plant, including all buildings and machinery at both ends, was sold for $10 000, leaving $110 000, or $9.73 per lin. ft., as the net overhead cost of the tunnel. The company purchased the plant and expects to use it to generate electricity to be transmitted to Greeley, Colo., and the machinery, other than that needed for electrical power development, is being taken away.

In all, 985 ft. of tunnel were timbered at a cost of $2.32 for material and $4.73 for labor. This work was done by the contractor at cost, plus 10 per cent.

During March, April, and May, 1911, 14 904 cars of muck, aggregating 9 107 cu. yd., were taken out. The total tunnel excavation, as called for in the contract, is 4 296 cu. yd., or a ratio of 2.11 cu. yd. measured in the cars to 1 cu. yd. paid for in place. This quantity includes all over break, which was not measured, therefore it does not represent the true swell in breaking up the rock.

In conclusion, the writer wishes to acknowledge his indebtedness to Mr. J. A. McIlwee for the information and figures he has furnished for the preparation of this discussion.

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PAPERS AND DISCUSSIONS

FAULTS IN THE THEORY OF FLEXURE.

Discussion.*

By Messrs. E. W. Stern, Theodore Belzner, and Henry S. Prichard.†

Eugene W. Stern, M. Am. Soc. C. E.—This admirable paper is very interesting, and the author discusses a number of points to which the speaker has given attention in his practice. The illustration of the diminution in strength of a channel due to unsymmetrical loading has been amply verified in the speaker's experience. An occurrence of many years ago has never been forgotten, as the results were so surprising: A line of shafting was supported on some 12-in. channels in which the factor of safety, if calculated by ordinary methods, was fully 20. Under the weight of the shafting and the pull of the belts, these channels actually bent over and collapsed. As very often happens when material leaves the mills, there was a slight initial bend in the webs of the channels, which had not been straightened in the fabricating shops, and, besides, their ends simply rested on girders without any provision for bracing them in an upright position. The fault was rectified by taking out the channels, straightening them, tying the ends together, and bracing them so that they could not bend sideways, and no further difficulty was ever experienced. Mr. Stern.

With reference to internal stresses in a beam due to cooling after being rolled, Mr. Prichard's remarks are confirmed by the speaker's experiences. Some years ago he had occasion to shear some 8-in. beams, 10 ft. long, down the middle of the web in order to make 4-in. T's out of them, and it was found that each half of the beam curved outward about 2½ in. This happened, not with one beam alone, but almost uniformly with about twenty.

* Continued from May, 1912, *Proceedings*.
† Author's closure.

Mr. Stern. Referring to the comparison between the strength of beams of the Standard and Bethlehem sections, given in Mr. Prichard's tables, the speaker would very much like to know under what conditions these tests were made and by whom, because he has been using a great many Bethlehem sections on account of their supposed economy over Standard sections. If they are not as strong as the Standard sections, he, of course, would like to know it. The only tests of which he has seen a record were those made by Professor Marburg, which are alluded to by Mr. Prichard. To the speaker's mind, these tests were of no value for the reason that they were made under conditions which, in all his practice, he has never seen in building or bridge construction. No attempt whatever was made to prevent the webs from buckling, nor to brace the ends of the beams laterally. These Marburg tests on the full-sized sections convinced the speaker of only one thing, namely, that they are of no service to the practicing engineer. Therefore, it would be of great interest to know just how these later tests, mentioned by the author, were made and by whom. It is hoped that Mr. Prichard will make a detailed statement of this matter.

Tests of full-sized sections for comparing the relative strengths of different designs will be of value only if exactly the same conditions are adopted for the testing of every piece, and likewise if the method of testing is devised in such a way as to impose conditions of loading which might commonly be obtained in practice. If the beams in one particular set were not straightened and trued up as carefully as those in another set, before testing, the latter, of course, would be expected to give higher values. The case of the channels, cited by the speaker, in which a little straightening of the webs and the use of a better type of end connections very materially strengthened them, emphasizes this point.

Another important point which Mr. Prichard has brought out is the necessity of using stiffeners in the webs of beams. Many beams have a slight curvature in the web when they come from the mill; likewise, the flanges are not exactly parallel. If, therefore, heavy concentrations are applied to the top flanges of beams, such as, for instance, in the grillage beams under columns, the crippling of the webs must be carefully guarded against, and, in such cases, it is the speaker's invariable practice to use stiffeners ground to fit tightly between the flanges, whether or not the calculations require them.

Referring to the theory of flexure as applicable to short beams, will the author be so kind as to state, according to his theory, what the strength would be of, say, a 20-in., 100-lb. beam, 6 ft. long, as compared with that given in the handbooks?

Mr. Belzner. THEODORE BELZNER, ASSOC. AM. SOC. C. E. (by letter).—Mr. Prichard states that many experiments have shown that the over-

straining of a beam lowers the elastic limit of the metal in it temporarily, and that this elastic limit returns after a rest. If such is the case, it would be very interesting to know when the normal elastic limit has again been reached, and also the length of time necessary for a complete recovery; for example, in the case of I-beams varying from 12 to 20 in. in depth. Mr. Belzner.

Henry S. Prichard, M. Am. Soc. C. E. (by letter).—Referring to Mr. Stern's discussion, it is gratifying to have the portions of the paper relating to the use of channels as beams, and to the initial internal stresses in I-beams, confirmed by Mr. Stern's experience. Mr. Prichard.

In regard to his query as to the strength of a 20-in., 100-lb. I-beam, 6 ft. long: By reasoning similar to that used in computing Table 1, the uniformly distributed load which would produce a given intensity of stress within the elastic limit in the extreme fibers of the beam cited, supposing it to be initially free from internal stresses, would be about 4% less than indicated by the theory of flexure. After the elastic limit is passed, however, the beam may develop additional permanent resistance by taking a slight permanent set, as described in Section 6 of the paper. The load for such a short span would probably be concentrated instead of uniformly distributed. If the loads are concentrated, the points of concentration and the manner of applying the loads would have to be specified before Mr. Stern's query could be answered.

The tests of the beams cited in Table 4 were part of a series of tests of beams, beam connections, and plate girders undertaken by the Carnegie Steel Company, the Illinois Steel Company, and the American Bridge Company, primarily for their own information, and conducted by a committee composed of James H. Edwards, John C. Neale, and John Brunner, Members, Am. Soc. C. E.

For the purpose of making the tests, the 2 000-ton eye-bar testing machine at the Ambridge plant of the American Bridge Company was reconstructed by fitting a framework against the main longitudinal girders, as shown by Plate LXXVIII. The pulling arrangement consisted of an equalizing device pulled by an eye-bar connected to the cylinders of the eye-bar testing machine, and carrying at its ends shackles, by the adjustment of which loads could be applied at different points. The construction of this testing machine required the placing of beams in a horizontal position, but, as the beams were supported and guided at intervals of one-third of the length, the bending moment in the vertical direction became a minimum, and was disregarded. The apparatus was arranged so that either one central load or two equal loads, symmetrically placed at varying distances apart, could be applied to the beam. The loads were applied by small increments, in accordance with schedules suited to the depth of the beam, its length, and the condition of loading. The sets and deflections under

Mr. Prichard. loading were read on circular deflectometers, which magnified the movement of the beam so as to permit of close readings. One of these deflectometers was placed at the center of the beam, or as closely thereto as possible, and two others at approximately equal distances on each side of the center, and usually as nearly as possible to the third points of the beam spans.

From the high standing of the engineers who conducted the tests, as well as from the purpose for which the tests were made, the writer is confident that every reasonable precaution was adopted to make them truly informative, actually and relatively.

The depths of the beams given in Table 4 were selected for the reason that they are those most commonly used in ordinary building construction. The lengths used in these tests were selected as a fair average of the spans for which these beams are used. The conditions of loading and supports were those usual in actual construction, and were as described on page 251.*

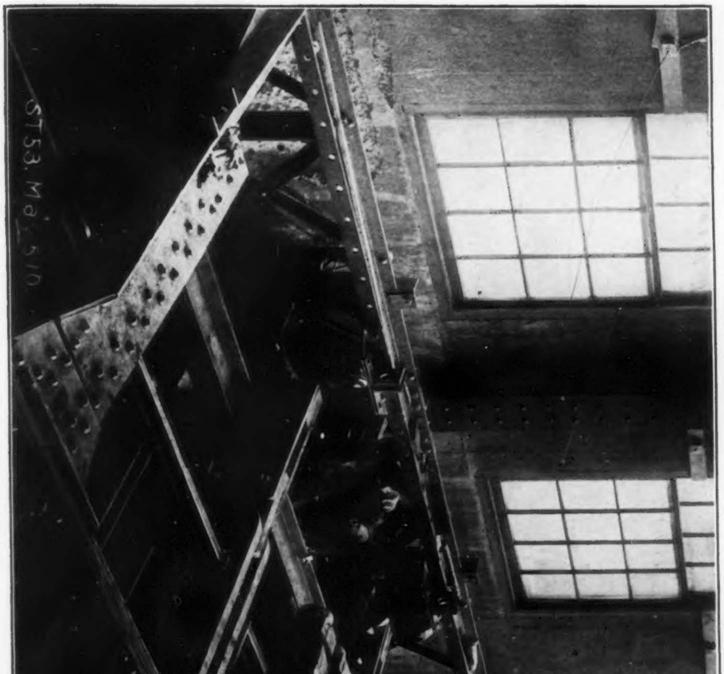
Referring to Mr. Worthington's discussion, if the theory of flexure was strictly accurate, it would apply rigidly to both steel and soft rubber. In a strict sense, however, as shown in the paper, it is only an approximation, though a remarkably close one within wide limits. As a theory, it has its faults, which, naturally, are more manifest in soft rubber than in steel. For this reason, soft rubber was chosen to exhibit the fact that originally straight cross-sections deform in a reverse curve, somewhat like a long $\int$, instead of remaining straight, as assumed in the theory. In the case of the rubber beam exhibited, this deformation is partly due to the local effects of concentrations, which local effects, as pointed out in Section 2 of the paper, are not taken into account in the ordinary theory of flexure; hence the rubber beam really illustrates a combination of faults described in Sections 1 and 2 instead of illustrating only the fault described in Section 1; and, though suggestive, is not in itself conclusive evidence of the fault described in Section 1. A demonstration of this fault, however, is given by the noted authority, Professor C. Bach,† and this fact is cited in the paper, in which also an independent proof is given. Mr. Worthington, who denies that there is any similarity between "this rubber beam" and "an actual beam of steel," does not mention either of these demonstrations, and it does not appear, from his discussion, whether he rejects (without argument) the fact demonstrated, or accepts the fact, namely, that originally plane cross-sections do not remain plane during flexure, and only objects to the means used to illustrate it.

In Section 2 it is shown that the ordinary theory of flexure neglects the local effects of concentrated loads and reactions. One of these effects is a tendency to buckle the web.

* *Proceedings*, Am. Soc. C. E., for March, 1912.

† "Elastizität und Festigkeit," p. 459.

PLATE LXXVIII.
PAPERS, AM. SOC. C. E.
AUGUST, 1912.
PRICHARD ON
FAULTS IN THE THEORY OF FLEXURE.



BEAM TESTING MACHINE, AS ARRANGED FOR THIRD POINT LOADING.

Mr. Prichard.

In the standard I-beam sections, adopted by the Association of American Steel Manufacturers in 1896, the webs are of such thickness that they are rarely the weakest point; in consequence, architects and engineers have given the webs little consideration, and have very seldom used stiffeners.

As beams with thinner webs than the standard are now rolled, the paper calls the special attention of architects and engineers to the fact, and points out that thin webs have less resistance to buckling than thick ones, and that, unless the web is reinforced, it has to be strong enough to resist both the shear and the concentrations, where there are any.

Even when there is no danger of buckling, it is an open question whether it is true economy to use beams with thinner webs than those of the standard beams.* Whether, when the web is too weak safely to stand the concentrations, it is best to use beams with thicker webs or to add "stiffeners milled to bear against the flanges and riveted directly to the web of the beam," as recommended by Mr. Worthington, depends on the conditions.

Mr. Worthington can see no reason, in theory or in fact, for making any distinction, as regards concentrations, between standard beams and those with thinner webs, but he adds the general caution that "the designer should always consider the matter of how each concentrated load shall be applied to the beam he is designing, whatever its section."

It is to be hoped that those who are inclined to heed the caution as to thin webs will not be misled by Mr. Worthington's discussion into the belief that there is only one company rolling beams with thinner webs than standard. There are, in the United States, at least two companies which roll such beams, and the practice may become general. The point with which the paper deals in this regard is the shapes of the beams. The names of the concerns rolling them is not material to the engineering questions discussed.

The caution as to thin webs occurs in Section 2. To Mr. Worthington, this section, three-fourths of which are entirely general, seems to be merely an invidious comparison between standard I-beams and those rolled by a particular company, and he makes the same mistake regarding the "whole gist" of the paper. To him the portions which deal with any theme other than the comparative value of the said beams, or which have any broader application, are few and unimportant.

The writer's intention to publish his investigations of faults in the theory of flexure was formed, and much of the matter published existed in a fragmentary state among his papers, before there was, in the United States, at least, any departure from the standard I-beams, in the way of thinner webs and flanges.

* In writing the above, transverse strength was the consideration in mind, but the condition of an I-beam just disclosed by the excavation for the removal of "The Hump" in Pittsburgh suggests other considerations. The beam is 8 in. deep and 8½ ft. span, and supported a sidewalk. Its web has entirely rusted through from connection angle to connection angle, leaving no connection whatever between the flanges for 8 ft.

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The manuscript, with the exception of the references on the last three pages to manufacturers' tests of steel **I**-beams, was written, almost exactly as presented, before the writer had any information as to the results of those tests, and it would have been published had he never heard of them. The general report of those who conducted the tests refers to, but does not incorporate, records of deflections and permanent sets of each beam as successive loads were applied during the progress of its test (the writer has not examined these records), and it gives diagrams showing the averages of the permanent sets observed from 0 to 0.4 in. for each size of beam, for each span, for each condition of loading.

From these graphic diagrams of permanent sets, as the paper states, the writer prepared Table 4, in which he endeavored to epitomize the results shown by the diagrams, by scaling the loads which produced permanent sets of 0.1 in. and 0.4 in. and converting the amounts scaled into multiples of the working load, W.

Those familiar with the preparation of such diagrams realize, of course, that it is not necessary to know in advance just what loads will produce given permanent sets, as assumed by Mr. Worthington. Instead, the method is to note the sets under successive loads, and connect the points indicating such sets by lines. Believing that Table 4 would be of some interest, that the large loads required to produce small permanent sets were reassuring after the poor showing made in Professor Marburg's tests, and that it was pertinent to a paper on flexure which criticized the adoption of new shapes of **I**-beams without first testing them, permission to publish this table in the paper was sought and obtained, notwithstanding that it was pointed out at the time that the paper advocated repeated and endurance tests as a scientific means of determining permanent capacity, and would make comment regarding the table accordingly.

The tests and report thereon were not made with any regard to the paper, and so, of course, no special observations were made for the purpose of illustrating it and testing its author's contentions, as Mr. Worthington by his questions implies should have been done; in fact, those who made the tests and the report had no knowledge that such a paper was in preparation or contemplated. On the other hand, the paper was not written for the purpose of publishing the tests.

Mr. Worthington makes two contentions relative to including in the paper the manufacturers' tests of **I**-beams. He leads up to the first contention by a supplement of his own to the theory of flexure which involves the undemonstrated and fallacious proposition that the loads which produce slight permanent sets, say 0.1 in. (in **I**-beams of the same depth and moment of inertia), are inversely proportional to the flange widths. He concludes, with regard to the beams in Table 4, that theoretically it should take 21 and 22% more load to produce

a slight permanent set in the standard than in the new beams, and, after comparing this result with the 18.6% difference shown by experiments in producing 0.1 in. of permanent set, he contends: Mr. Prichard.

"Certainly such close agreement between theory and tests should bar out this table from a paper intended to demonstrate faults in that theory."

Mr. Worthington, before giving his second contention, points out the additional data which he considers should have been included in the table, and alleges that either those who made the tests were incapable or the data were purposely withheld, and he then contends:

"For surely it is not a tenable hypothesis that the author did not appreciate the overwhelming importance to his paper of including all these measurements in his record of those tests."

Why it is of overwhelming importance to a paper to include certain data in a table which should certainly be "barred out" of that paper Mr. Worthington does not explain.

The writer protests against Mr. Worthington's claim that the theory of flexure indicates that it takes 22 and 21% more load to produce a slight permanent set in 12 and 15-in. standard I-beams than it does to produce the same set in the corresponding beams of new shapes. It is surprising that one who objects so strongly to having greater resistance to concentrated loads claimed for the webs of the standard beams, on account of their greater thickness, should claim much greater theoretical resistance to permanent set for standard beams than for those of the new shapes shown in Fig. 8 C. Of course, there must be some good reason for the smaller resistance to permanent set shown in Table 4 for the new shapes, but it has not been accounted for by the theory of flexure, and can be only partly explained, if at all, by Professor James Thomson's theory of overstrained beams (endorsed by his brother, Lord Kelvin). Possibly, it may be due, in part at least, to the tendency, which naturally develops as beams deflect, of the edges of the flanges to lag behind their centers in taking their share of the strain; a tendency which is naturally greatest in wide thin flanges.*

The principle of the first contention, namely, that facts which tend to confirm a theory should certainly be omitted from a paper intended to demonstrate faults in that theory and show its limitations, is wholly wrong. Engineers, in their papers and discussions, should not be partisan advocates, or attorneys pleading some client's case, but sincere contributors to engineering knowledge who endeavor to be absolutely fair.

The second contention overstates the importance, to a paper on faults in the theory of flexure, of complete details of the tests recorded

* The edges of the top flange tend to buckle downward and the edges of the bottom flange to straighten upward, as a beam deflects, and thereby relieve themselves of stress, and they should thus relieve themselves to the extent permitted by the cross-sectional stiffness of the flanges, thus compelling the centers of the flanges, which cannot relieve themselves in that way, to take more than their share of the stress.

Mr. Prichard. in Table 4. It is doubtless a fact, however, that a paper which dealt exclusively with I-beams and gave complete details of these tests would be of greater immediate interest to engineers than the paper under discussion. The writer would be pleased to have such a paper presented to the Society by the engineers who conducted the tests, and whose efforts he rates very highly.

The paragraph in the paper which called forth Mr. Worthington's statement as to the writer's opinion of those who conducted the tests is as follows:

"On an average, it took 18.6% more load to produce a permanent set of 0.1 in. in the beams of standard shape than in the nominally equivalent beams of new shapes, and 8% more to produce a permanent set of 0.4 in. Whether or not this indicates a corresponding superiority in permanent capacity, what the permanent capacities are, and what permanent sets the beams would take under their maximum permanent loads, are questions to be decided by scientific experiments."

Those who conducted the tests followed the very general practice of making but a single test of each piece. The writer favors repeated and endurance tests for determining maximum permanent capacity. If he had a poor opinion of those who follow the method of making a single test of each piece, it would have to include nearly the entire Profession.

The paragraph above quoted contains the only comment in the entire paper on the results of the tests given in Table 4, yet Mr. Worthington uses the phrase: "the author's 18.6% of implied advantage," and states that the author assumes such a superiority to be shown by the data published in his paper. Mr. Worthington must have derived his impression as to superiority of the standard beams over those of new shapes from the results of the tests themselves and not from any comment by the writer.

Mr. Worthington overestimates the comparative importance of ultimate strength, deflections, and measured strains; he greatly underestimates the importance of permanent sets, which he regards as "comparatively valueless," and he expresses a purely theoretical and wholly inadequate conception of elasticity and its limits, as applied to materials of construction.

"Elasticity is that property of matter by virtue of which a body will not change in bulk or shape except by force, and will recover its original bulk or shape on the removal of the force."*

For the purposes of the structural engineer, it is the most useful property of matter; the one on which he relies for the permanent strength of the structures he designs, and on which he bases his theories of deformation, and distribution of stress.

It would seem that all that is necessary is to commit to memory the theories, to ascertain the limits to elasticity of the materials of con-

* "Elasticity and Fatigue of Wrought Iron and Steel," by Henry S. Prichard, *Industrial Engineering*, April, 1909, p. 15.

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struction, and to take a course in mathematics; then structural engineering becomes a mere matter of computation. This is the conception which students in structural engineering are apt to derive from their textbooks, and which some carry with them into their engineering practice. It is a beautiful conception, and, if it was a correct one, and if the limits to elasticity could be noted and recorded, "with great ease," the engineer who failed to note the elastic limit in making tests or who omitted it from his published records of the tests would indeed be delinquent.

This theoretical conception of ideal materials used only within the limits of perfect elasticity, however, does not harmonize with the facts as to the actual materials with which the engineer has to deal. Consider, for instance, the tests of eye-bars at the Watertown Arsenal* given in Table 5:

TABLE 5.—Compiled from Tests of Eye-Bars at the Watertown Arsenal.

Test No.	Material.	Normal size, in inches.	Load, in pounds per square inch.	Gauged length, in inches.	Permanent set, in inches.
4134	Steel.	5 × 1	5 000	260	Not recorded.
4135	"	5 × 1	5 000	260	0.0059
4136	"	5 × 1	5 000	260	0.0065
4137	"	5 × 1	5 000	260	0.0010
4138	"	5 × 1	5 000	260	0.
4139	"	5 × 1	5 000	260	0.0051
763	Wrought iron.	5 × 1⅛	5 000	180	0.0034
764	" "	5 × 1⅛	5 000	180	0.0006
765	" "	5 × 1⅛	5 000	180	0.0055
766	" "	5 × 1⅛	5 000	180	0.0031
767	" "	6 × 1½	5 000	180	0.0006
768	" "	6 × 1½	5 000	180	0.0003

In all the cases in Table 5 the first reading, after the micrometer was set at zero for the initial load of 1 000 lb. per sq. in., was at 5 000 lb. per sq. in.; thus the limit to perfect elasticity indicated for most of the bars was less than 5 000 lb. per sq. in. Recent tests by manufacturers exhibit similar results.

These permanent sets in eye-bars under low loads are small, and were appreciable only by reason of long gauged lengths and fine micrometric precision. In eye-bars and in smaller pieces of wrought iron and steel, when the direct load is increased slowly, a condition is eventually developed in which a great increase in elongation (or linear compression in crushing tests) occurs with comparatively little increase in load. The point at which this marked change in deformation occurs is properly termed the yield point, and is often called the elastic limit. It is well illustrated in a test of an eye-bar, made for the late George S. Morison, Past-President, Am. Soc. C. E., at the Watertown Arsenal,† and given in Table 6.

* Report for 1886, Part 2, pp. 1569-1617.

† Report for 1901, p. 410.

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TABLE 6.—TEST OF EYE-BAR FOR GEORGE S. MORISON.
Gauged Length, 160 Inches.

Load, in pounds per square inch.	Elongation, in inches.	Permanent set, in inches.	Remarks.
1 000	0.0000	0.0000	Initial load.
5 000	0.0244	0.0005	
10 000	0.0529	0.0008	
20 000	0.1090	0.0017	
25 000	0.1375	0.0035	
28 000	0.1562	....	Called "Elastic Limit."
29 000	0.1653	....	
30 000	0.1830	0.0220	
31 000	0.2222	....	Yield point. (Term not used in report of test.)
32 000	0.7800 0.8220	0.6461	Elongation 2 sec. later.
		Percentage of elongation	
33 000	1.03	0.64	
34 000	2.10	1.31	
35 000	2.97	1.86	
36 000	3.22	2.01	
37 000	3.44	2.15	
38 000	3.72	2.32	
39 000	3.98	2.49	
40 000	4.35	2.72	
41 000	4.67	2.92	
57 730	....	....	Tensile strength.
0	23.59	14.7	

Contraction of area, 53.5 per cent.

The yield point is difficult to observe accurately in making rapid tests of small pieces, and, usually, is not sharply marked in hard steel, in steel which has been worked cold, or in bending tests.

To illustrate the uncertainty as to the precise position of the yield point in bending tests of I-beams, the U. S. Board's Tables I, II, IV, and XI of tests of wrought-iron I-beams are here reproduced.

These tests were made under the direction of, and are given in a report signed by, the noted civil engineers, William Sooy Smith, M. Am. Soc. C. E., and the late Q. A. Gillmore, M. Am. Soc. C. E., Lt-Col. of Engineers, Brevet Major-General, U. S. A.* The Report states, among other things:

"The loads were increased in each case until unmistakable signs of the failure of the beam appeared.

"The indications which have been relied upon for determining the elastic limit are as follows:

"1st. An unusual increase in the increments of deflection per 1 000 pounds and a corresponding decrease in E.

"2d. The set becoming appreciable and beginning to increase rapidly.

"3d. Often by a point of contrary flexure in the deflection curve, which becomes particularly noticeable when the curve of difference (G-S) is platted.

"Lastly, a general inspection of the diagram itself."

* Report of U. S. Board on Testing Materials, 1881, Vol. 2, p. 215.

U. S. Board's Table I, 1881, Vol. 2, Page 226.

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15-in. Beam. Clear span, 20 ft. Length, 20 ft. 10⅛ in. Total weight, 1 012.5 lb. Weight per yard, 145.73 lb. Moment of Inertia, 536.56.

Number of trial.	Weight applied at center, in pounds.	Deflection, in inches.	Deflection, in inches per 1 000 lb.	Permanent set, in inches.	Permanent set, in inches per 1 000 lb.	Lateral deflection.	Coefficient of elasticity, E.
1........	9 306	0.185	0.0187	0.00	0.00	....	38 310 000
2........	12 237	0.24	0.0187	0.00	0.00	....	38 275 000
3........	15 566	0.33	0.0204	0.015	0.0009	....	35 021 000
4........	17 722	0.395	0.0216	0.035	0.0017	....	33 160 000
5........	19 586	0.46	0.0227	0.045	0.0022	....	31 362 000
6........	21 225	0.525	0.0241	0.055	0.0025	....	29 843 000
7........	22 351	0.56	0.0243	0.095	0.0041	....	29 376 000

Remarks.—The limit of elasticity does not appear to have been reached in this test, so far as can be discovered from the columns of sets and deflections, or from the diagram, unless, indeed, it is reached in No. 7.

The unit strain, f, for No. 7 is 22 400 lb., which appears to be too low for an elastic limit with a beam which has so high a modulus of elasticity.

$$E.\ L. = 22\,351 \text{ plus } 506.$$
$$f = 22\,400.$$
$$E_m = 33\,621\,000.$$

U. S. Board's Table II, 1881, Vol. 2, Page 228.

10½-in. I-Beam. Clear span, 22 ft. Length, 23 ft. 8½ in. Total weight, 1 033 lb. Weight per yard, 130.68 lb. Moment of Inertia, 221.86.

Number of trial.	Weight applied at center, in pounds.	Deflection, in inches.	Deflection, in inches per 1 000 lb.	Permanent set, in inches.	Permanent set, in inches per 1 000 lb.	Lateral deflection.	Coefficient of elasticity, E.
1........	6 343	0.445	0.0641	0.0	0.0	0.0	26 960 700
2........	7 010	0.490	0.0645	0.0	0.0	0.0	26 836 900
3........	7 728	0.510	0.0614	0.0	0.0	0.0	28 217 200
4........	8 362	0.590	0.0654	0.0	0.0	0.0	26 264 900
5........	8 827	0.610	0.0649	0.0	0.0	0.0	26 704 700
6........	9 285	0.630	0.0638	C.0	0.0	0.0	27 113 200
6........	9 285	0.650	......	0.03	0.0031	0.0	....
7........	9 683	0.670	0.0650	0.03	0.0023	0.0	26 520 900
8........	10 138	0.710	0·0663	0.03	0.0023	0.0	26 134 100
9........	10 541	0.740	0.0664	0.03	0.0027	0.0	26 015 700
10........	10 940	0.770	0.0667	0.04	0.0034	0.0	25 897 500
11........	11 476	0.810	0.0670	0.05	0.0041	0.0	25 762 000
12........	12 216	0.845	0.0660	0.050	0.0040	0.0	26 208 200
13........	12 895	0.905	0.0670	0.065	0.0048	0.0	25 767 200
14........	13 831	0.960	0.0667	0.065	0.0045	0.0	25 975 600
15........	14 611	1.010	0.0664	0.065	0.0042	0.0	26 024 400
16........	15 543	1.085	0.0674	0.065	0.0040	0.0	25 709 800
17........	16 675	1.195	0.0690	0.070	0.0041	0.0	24 981 400
18........	17 701	1.280	0.0700	......	......	0.0	24 706 300
18........	17 701	1.320	......	0.105	0.0056	0.0	....
19........	18 647	1.375	0.0715	0.120	0.0063	0.0	24 188 200
20........	19 565	1.505	0.0746	0.260	0.0130	⅛ in.	23 152 900
21........	20 236	1.645	0.0791	0.320	0.0150	½ in.	21 887 300
22........	20 725	1.770	0.0831	0.370	0.0176	¾ in.	20 819 000
23..	21 204	.....	......	.....	......	...	....
24........	22 436	2.465	0.107	1.04	0.0450	...	16 148 600

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Remarks.—The beam could not be broken vertically, as it yielded by buckling laterally.

Trial No. 6 was continued thirty-nine hours.

Trial No. 24 was repeated, the second application of the load causing the beam to bend sidewise.

The elastic limit appears, both from the table and from the curve diagram, to have been reached at Trial No. 19 with a load of 18 647 lb., as here the columns of reduced deflections and sets show the first decided irregularities and abnormal increase. The diagrams show this still more clearly.

To the applied load, 18 647 lb., must be added half the weight of the beam between the bearings, *viz.,* 477 lb. for the total concentrated load, namely, 19 124 lb. This equals a distributed load of 38 248 lb.

$$E.\ L. = 18\,647 \text{ plus } 477.$$
$$f = 29\,866.$$
$$E_m = 26\,099\,400.$$

The deflections from additional loads do not seem to be affected by the previous repeated loads, although left on for some time.

U. S. BOARD'S TABLE IV, 1881, VOL. 2, PAGE 232.

$10\frac{1}{2}$-in. Beam. Clear span, 22 ft. Length, 29 ft. 3 in. Total weight, 1 030 lb. Weight per yard, 105.63 lb. Moment of Inertia, 174.75.

Number of trial.	Weight applied at center, in pounds.	Deflection, in inches.	Deflection, in inches per 1 000 lb.	Permanent set, in inches.	Permanent set, in inches per 1 000 lb.	Lateral deflection.	Coefficient of elasticity, *E*.
1........	4 325	0.325	0.0677	0.00	0.0	....	32 587 000
2........	6 301	0.475	0.0701	0.00	0.0	....	31 472 000
3........	6 968	0.525	0.0705	0.00	0.0	....	31 220 000
4........	7 686	0.585	0.0716	0.005	0.0005	....	30 702 000
5........	8 320	0.635	0.0722	0.005	0.0005	....	30 504 000
6........	8 785	0.675	0.0728	0.015	0.0016	....	30 176 000
7........	9 243	0.715	0.0735	0.015	0.0015	....	29 229 000
8........	9 641	0.735	0.0728	0.015	0.0014	....	30 337 000
9........	10 096	0.765	0.0722	0.015	0.0014	....	30 425 000
10........	10 499	0.800	0·0727	0.015	0.0013	....	30 202 000
11........	10 898	0.835	0.0732	0.015	0.0013	....	29 986 000
12........	11 434	0.89	0.0745	0.025	0.0021	....	20 462 000
12........	11 434	0.90	....	0.045	0.0037	....	....
13........	12 174	0.96	0.0761	0.045	0.0035	....	29 013 000
14........	12 853	1.00	0.0752	0.045	0.0033	....	29 341 000
15........	13 798	1.07	0.0748	0.045	0.0031	....	29 376 000
16........	14 578	1.17	0.0780	0.075	0.0050	....	28 270 000
17........	15 501	1.31	0.0820	0.115	0.0071	....	26 836 000
17........	15 501	1.32	....	0.155	0.0097	....	....

Remarks.—Elastic limit at No. 15:

$$E.\ L. = 13\,793 \text{ plus } 397.$$
$$f = 28\,221.$$
$$E_m = 30\,270\,100.$$

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Average of Tests 2 and 3, beams same size and span:

$$E = 29\,409\,000.$$
$$f = 27\,159 \text{ lb.}$$

Average load at elastic limit = 13 270 plus 390.

U. S. Board's Table XI, 1881, Vol. 2, Page 244.

8-in. Beam. Clear span, 14 ft. Length, 16 ft. 5¾ in. Total weight, 353 lb. Weight per yard, 64.29 lb. Moment of Inertia, 62.34.

Number of trial.	Weight applied at center, in pounds.	Deflection, in inches.	Deflection, in inches per 1 000 lb.	Permanent set, in inches.	Permanent set, in inches per 1 000 lb.	Lateral deflection.	Coefficient of elasticity, E.
1........	4 189	0.255	0.058	0.005	0.001	....	27 199 000
2........	5 351	0.335	0.060	0.005	0.0009	....	26 196 000
3........	6 364	0.380	0.058	0.015	0.0023	....	27 420 000
4........	7 749	0.460	0.058	0.015	0.0019	....	27 275 000
5........	8 848	0.535	0.059	0.020	0.0022	....	26 773 000
6........	10 159	0.615	0.059	0.030	0.0029	....	26 663 000
7........	10 961	0.660	0.059	0.045	0.0040	....	26 773 000
8........	12 237	0.740	0.059	0.070	0.0056	....	26 611 000
9........	13 851	0.930	0.066	0.155	0.011	....	23 915 000

Remarks.—The limit of elasticity is not so clear in the 8-in. beam as could be desired, and its determination is largely a matter of judgment.

Limit of elasticity assumed at No. 8.

$$E.\ L. = 12\,237 \text{ plus } 165.$$
$$f = 33\,957.$$
$$E_m = 26\,863\,750.$$

"It would seem as if too little attention has heretofore been given to *set*, as the data referring to it are very meagre as compared with that on deflection."

In only seven of the twenty-six beams tested by the U. S. Board is the elastic limit stated without qualification. It is evident that what they endeavored to determine was not the limit to perfect elasticity but a point corresponding as nearly as might be to the yield point in tension tests, that it was not well marked, and that it was nominally established somewhat arbitrarily.

To revert to the manufacturers' tests of I-beams epitomized in Table 4: Even if it had been possible to obtain, from diagrams given in the report of these tests, the points at which the first minute permanent sets occurred, it would not have been proper to have published these points as "the elastic limit," in view of the prevalent use of this term to indicate the yield point, any more than it would have been proper for the Watertown Arsenal to have indicated the

Mr. Prichard. elastic limit of the eye-bar for Mr. Morison (Table 6) as 5 000 lb. or less.

The writer could have obtained and published so-called elastic limits by a general inspection of the diagrams and by noting where the permanent set began to increase rapidly, as was done by Smith and Gillmore, and, had these limits been thus obtained and published, the average of the elastic limits thus published for the standard I-beams would have been in excess of the average for the new shapes by a percentage not far from, but somewhat greater than, the 18.6% by which the loads required to produce 0.1 in. of permanent set in standard beams exceeded those required to produce the same set in the new shapes. Such a determination, however, would of necessity have been in most cases arbitrary and subject to whatever influence the writer's prejudice, if he had any, might have exerted. For these reasons the elastic limits were not published.

It is explained in the paper that the seemingly low limits to elasticity observed in many experiments are due to internal stresses; and the investigations of Thomson, Thurston, and Bauschinger are referred to as showing this fact. The real elastic limit of the material in such cases is the computed stress plus the initial internal stress. It is further pointed out in the paper that the investigations of the authorities cited indicate that overstrained iron and steel recover their elasticity after a rest, that the yield point is raised by loading, and that the elasticity is perfected up to the amount of the load within limits somewhat in excess of the original yield point.

Those who hold with Mr. Worthington that there is a well-marked, easily-determined limit to perfect elasticity and that "just as soon as the extreme fibers of a beam are stressed beyond the elastic limit of the material, the beam has failed and is no longer useful in the art of construction," would do well to familiarize themselves with swaging, cold-rolling, wire-drawing, and rod-twisting, none of which would be possible if overstraining permanently destroyed the elasticity of the material. They should also consider the effects of punching, plate and shape straightening, and other shop operations which would weaken the material to such an extent as to destroy its usefulness, if it could not recover. They should then consider the internal stresses developed in the cooling of castings, beams, etc.

A good illustration of the raising of the yield point and the perfecting of elasticity is afforded by a test of an eye-bar at the Watertown Arsenal, as condensed in Table 7.

In eye-bars, the great elongation at their yield points destroys their usefulness as members of a structure after this point is reached; in compression members, unless they are very short and solid or compact, failure will take place from buckling or local crippling at the yield point or before it is reached; but it appears from the investigations

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and tests cited that it is reasonable to expect that I-beams, when supported laterally, can have their elasticity perfected and their permanent strength enhanced, without undue deformation, by overstraining, unless they have been rolled so thin that their resistance to local crippling or other influence of attenuation is the determining factor.

It is not well, however, to assume a permanent increase, in the strength of the material of which I-beams are composed, much beyond its original yield point, as Bauschinger's experiments have shown, for material so strained that there is, as time goes on, a gradual yielding; besides, there is the danger of an insidious fatigue by the gradual extension, in the hardened material, under intense stresses, of microscopic flaws into planes of rupture, with little warning in the way of deformation.

If the elasticity of steel can be perfected up to the yield point by overstraining, and if the material has not been rolled too thin, I-beams, according to the theory explained by Professor Thomson and adopted by his brother, Lord Kelvin, in his article on elasticity,* can have their permanent capacity in one direction raised above that computed by the ordinary theory of flexure without straining the material appreciably beyond its original yield point, and without undue permanent set. Its elasticity under a load in the opposite direction, however, according to this theory, would be reduced.

If one of the 15-in. standard I-beams, tested as described in the paper, was loaded until the horizontal layer of material half way between the neutral axis and the extreme fiber was strained to the yield point, the material between this layer and the extreme fiber would be strained very little above the yield point, and the resistance of the beam at this stage of the test would, in consequence, be about 15% greater than it was when the extreme fiber first reached the yield point.† If this critical load should now be removed and the beam be allowed to rest, or even if it rested with the load on, it would thereafter, according to Thomson's theory, withstand any subsequent application of the load in the same direction without any additional permanent set.

As the yield point of the flanges of these standard 15-in. beams was a little more than 38 000 lb. per sq. in., and the working load is based on 16 000 lb. per sq. in., the permanent capacity thus indicated is about $2\frac{3}{4}$ times the working load $\left(W \dfrac{38\ 000}{16\ 000} \times 1.15\%\right)$. For the new shapes of 15-in. beams tested, using the higher yield point shown in the specimen tests of the flanges (40 000 lb.), the result is the same;

* Encyclopedia Britannica, 9th Edition, p. 798.

† The yield points in tension and compression in steel are about the same, and are here so assumed. A layer quite close to the neutral axis could be assumed as the one strained to the yield point and it would make very little increase in the computed result, but the assumption that the material between this layer and the extreme fiber would be strained very little above the yield point would not then be correct.

Mr. Prichard. $2\frac{3}{4}$ times the working load. This theory, however, assumes that the metal is sufficiently compact practically to avoid the weakening influence of attenuation; an influence difficult to analyze and only satisfactorily determinable by experimental investigation.

TABLE 7.—TEST NO. 4136 OF STEEL EYE-BAR (NOMINAL SIZE, 5 BY 1 IN.) AT THE WATERTOWN ARSENAL, REPORT FOR 1886, P. 1578.

Load, in pounds per square inch.	IN 260 INCHES.		Elongation from center to center of pins (25 ft. 8 in.), in inches	Remarks.
	Elongation, in inches.	Set, in inches.		
1 000	0.	0.	0.	Initial load.
5 000	0.0456	0.0065		
10 000	0.0915	0.0085		
15 000	0.1369	0.0089		
20 000	0.1815	0.0096		
25 000	0.2264	0.0101		
30 000	0.2720	0.0109		
.....	......	......		
37 330	0.3459	......	......	Called "Elastic Limit."
37 710	0.3700	......	......	
38 100	0.5665	......	......	(Yield point.)*
39 000	1.07			
40 000	2.35	......	2.76	
41 000	......	......	3.30	
42 000	......	......	3.61	
43 000	......	......	4.02	
56 000	......	......	10.70	
......	.....	......	.....	
64 000	......	......	18.35	
65 000	......	......	19.83	
65 750	......	......	21.05	
0	......	......	.....	Rested 5 minutes.
66 000	......	......	21.35	
70 286	......	30.42	36.60	Maximum load reached.
	In 290.42 Inches.			
	Elongation.	Set.	......	Rest—Duration not stated.
1 117	0.	0.	......	Micrometer reset.
39 095	0.4236			
55 850	0.6310	— 0.0010	......	Note the minus sign.
2 013	.....	......	......	Rested 1 hour.
1 117	......	— 0.0150		
1 117	0.	0.	..... .	Micrometer reset.
		0.0028	.. ...	Immediate set.
55 850	0.6263	0.0005	......	Set after 10 minutes.
		0.0000	......	Set after 12 minutes.
1 117	0.	0.		
16 755	0.1685	. ....	......	Rested 15 minutes under load.
72 605	0.8660	......	......	(Yield point.)*
72 605	0.8750	0.0301	......	Immediate set.
		0.0215	......	Set after 20 minutes.

* The term "yield point" does not appear in the report of the test.

The permanent set can also be indicated by the theory of overstrained beams, but not, even relatively, by any mere comparison of flange widths. Instead, it involves the consideration of stress and

strain in each element of the beam throughout its entire length, and is a very complicated problem. Mr. Prichard.

The time required to regain elasticity after overstraining is a subject of inquiry by Mr. Belzner. It probably depends on the size of the piece and the severity of the overstrain. At first the recovery is rapid, then it goes on gradually for hours and, possibly, for days. Overstraining wrought iron and steel makes the metal partly plastic. The viscosity of the plastic portion retards the tendency of the strictly solid portion to regain its shape. The hardening of the plastic portion is accompanied by an increase in tensile strength. Until the strength ceases to increase, the plastic portion has not completely hardened. The U. S. Board investigated the increase in tensile strength of overstrained wrought iron, with the following results.*

Average gain in	less than 1 hour	1.1%	(5 tests)
	less than 8 hours and more than 1 hour	3.8%	(8 tests)
	1 day	8.9%	(5 tests)
	3 days	16.2%	(10 tests)
	8 days	17.8%	(2 tests)
	6 months	17.9%	(12 tests)

The U. S. Board's Table II shows that Trial No. 6 was continued 39 hours. The writer knows of only one other endurance test. It was made by a U. S. Government engineer on a pair of 15-in. light iron I-beams (50 lb. per ft.) some years before 1884, by applying a uniformly distributed load on a 21-ft. clear span; the account† is as follows:

"They showed no signs of breaking with the maximum load applied, but could not be loaded further as they had deflected so that one of them touched the ground. In testing these beams, a load equal to twice the safe load‡ was first applied and allowed to remain on the beam 23 days; during this time the deflection increased from 0.98 inches to 1.12 inches. The load was then increased up to three times the safe load, and allowed to remain 15 hours, in which time the deflection increased from 2.01 to 2.09 in.; the load was then increased to 90 000 lb. [3.6 times the safe load] with a deflection of 2.7 in., which, after 18 hours, had increased to 2.77 in. The load was allowed to remain on the beams 15 days, when it was removed."

Referring to Mr. Vilar y Boy's discussion: The introduction of empirical formulas for flexure, based on breaking loads and used with a large factor of safety, marked an advance in engineering; and the high regard for experiments by the men who introduced these formulas is an example for modern engineers; but it is the permanent strength, rather than the temporary strength shown by the immediate breaking

* Report of U. S. Board for Testing Materials, 1881, Vol. 1, pp. 107-111.

† As given in the New Jersey Steel and Iron Company's "Book of Useful Information."

‡ Based on 12 000 lb. per sq. in., extreme fiber stress.

Mr. Prichard.

load, that should be investigated; and, as far as practicable, correct theory should be used in formulating the results of these investigations.

Referring to Mr. Dunham's discussion: In general, for the same quality of steel, when one section is made smaller than another by greater reduction in rolling, it should, theoretically, and does, usually, have a higher yield point and greater ultimate strength, and this fact should be properly considered in comparing the effects of overstraining in different sections.

The theory of overstrained beams applies to all symmetrical sections in which the metal is not too attenuated, and, therefore, should apply to rails. When compact symmetrical rolled beams, including I and rail sections, are moderately straightened by forces applied in the same direction and with the same distribution as the loads to which the beams will subsequently be subjected, theory indicates that their elasticity in the direction of the loads will be improved and its limit raised. If the amounts which the beams have to be straightened are great, or if the straightening forces and subsequent loads differ more or less in direction or distribution, the problem is complicated, and the great impact to which rails are subjected in service introduces further complication, as do other elements.

The recuperative and adaptive power of steel is a real reliance against the effects of the necessary operations of the mill and shop after the material leaves the rolls, but it is too much to expect that the net result will always be a gain in strength. The products of the mill and structural shop should be used with a reasonable margin of safety.

Theory is useful in suggesting lines for experimental investigation, in interpreting results of experiments, and in giving to these results their widest application, but intelligent observation and experiments should take precedence; they are what Mr. Dunham terms "real evidence."

MEMOIRS OF DECEASED MEMBERS

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

LEWIS KINGMAN, M. Am. Soc. C. E.*

DIED JANUARY 23D, 1912.

Lewis Kingman was born on February 26th, 1845, at North Bridgewater, Plymouth County, Mass., near the landing of the Pilgrim Fathers. He was the first of six children born to Isaac Kingman and Sibil Ames. He died in the City of Mexico, of pneumonia, on January 23d, 1912.

His early youth was spent at home. His education was in the common schools, supplemented by three and one-half years in Hunt's Academy, a local institution of merit, in which he finished his work in the winter of 1861.

In September, 1862, he entered on a three-year course in Civil Engineering with the noted firm, Shedd and Edson, of Boston, Mass. For this instruction he was to pay $100 per year. During his course with Shedd and Edson, which was reduced to one and one-half years by mutual consent, he lived at home, twenty miles from Boston *via* the Old Colony Railroad, and on this road he made daily round trips on student's tickets purchased at the rate of $52 per year.

During the course under Shedd and Edson he had a varied and valuable experience in municipal and corporate engineering.

For about four years, 1864-1868, Mr. Kingman was engaged in the Pennsylvania oil fields, with a brief period of about two months in engineering work in New York City. He was in the Pennsylvania oil fields when, in June, 1866, oil sold for $7 per barrel, and at 90 cents per barrel less than one year later.

In 1868 he went to St. Louis—attracted by railway building then in progress—and on July 13th engaged with the Eastern Division of the Atlantic and Pacific Railroad. He was with the Eastern and Western Divisions of that enterprise, in various capacities on construction and surveys, until October, 1871, when his party was disbanded at Las Vegas, N. Mex.

It is interesting here to note that, during this period, Mr. Kingman and the late James Dun, M. Am. Soc. C. E., had adjoining residencies on the construction of the Atlantic and Pacific Eastern Division, and that, later, Mr. Dun was Chief Engineer of the Eastern Division, known as the St. Louis and San Francisco Railroad, with headquarters at St. Louis, Mo., and Mr. Kingman was Chief Engineer of

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* Memoir prepared by A. A. Robinson and C. A. Morse, Members, Am. Soc. C. E.

the Western Division, known as the Atlantic and Pacific Railway, with headquarters at Albuquerque, N. Mex.

From the latter part of 1871 to the middle of 1877 Mr. Kingman was on a survey from Kit Carson, Colo., to Cimarron, N. Mex., in the interest of the Maxwell Land Grant Company, owner of a large Spanish land grant located in Colorado and New Mexico, and on Government land surveys, and other engineering work and commercial business.

In June, 1877, Mr. Kingman began work on the Atchison, Topeka and Santa Fé Railway under A. A. Robinson, M. Am. Soc. C. E., Chief Engineer of that Railway, and was employed in various localities, on mountain surveys and construction, in Colorado, New Mexico, and Arizona, until July, 1880, at which time he was on the Atlantic and Pacific Railway when Mr. Robinson, who had charge as Chief Engineer of the Atlantic and Pacific Railway as an auxiliary to the Santa Fé, withdrew from that enterprise. Mr. Kingman remained with the Atlantic and Pacific Railway until April, 1883; he was appointed Chief Engineer of this Company on January 1st, 1882, succeeding the late H. R. Holbrook, M. Am. Soc. C. E.

Mr. Kingman then (April, 1883) accepted the position of Chief Engineer of the Northern Division of the Mexican Central Railway, under Mr. Thomas Nickerson, President, and served in that capacity until June 1st, 1884, during which time he constructed 469 miles of the Mexican Central Railway.

In July, 1884, Mr. Kingman returned to the service of the Atchison, Topeka and Santa Fé Railway, under Mr. Robinson, Chief Engineer, and remained in the employ of that Company, most of the time as Assistant Chief Engineer, until January 1st, 1889.

During the years of active construction by the Santa Fé, Mr. Kingman built, in the years 1886 and 1887, 1 353 miles of railway, somewhat more than 2 miles for each working day—certainly a record which few engineers have exceeded. Early in 1889 Mr. Kingman was appointed City Engineer by the Mayor of the City of Topeka, Kans., and served under three mayors in that capacity.

In May, 1894, Mr. Kingman became connected with the Engineering Department of the Mexican Central Railway Company, under Mr. A. A. Robinson, President of that Company, and, shortly after that was made Chief Engineer, which position he held for nearly twelve years.

When the Mexican Central Railway was taken over by the Mexican Government in 1907 and became a part of the National Railways of Mexico, Mr. Kingman took the position of Engineer of Maintenance of Way, later becoming Office Engineer, as the former position required too active a life. Mr. Kingman retained this office until his

death, having been at his desk less than twenty-four hours prior thereto.

Mexico City was Mr. Kingman's headquarters during his connection with Mexican Railways, and he built 994 miles of railway in various parts of Mexico connecting with the Mexican Central System. At the time of the last reorganization of the Santa Fé System he was selected as one of a committee of three to appraise the Atlantic and Pacific Railway.

Mr. Kingman was a man of few words, great energy and determination, and a kind heart; a man of uncompromising honesty, who could be trusted with any responsibility he would accept, with entire confidence that the interests of his employers would be conserved under all circumstances; at the same time, he had a judicial mind, so necessary to every successful engineer.

His assistants and subordinates held him in high esteem for his sterling qualities; he was respected by contractors for his fair dealings and just decisions, and this is no small tribute when one recalls the many millions spent under his control during an unusually active and useful life.

Mr. Kingman was elected a Member of the American Society of Civil Engineers on July 1st, 1885, was a charter member of the American Railway Engineering and Maintenance of Way Association, a member of the National Geographical Society, the American Academy of Political and Social Science, of the Franklin Institute, and of the Masonic Fraternity.

Mr. Kingman is survived by a widow and five children now residing in Topeka, Kans. His oldest son was a student in the Kansas State University at the time of his father's death.

EDWARD PAYSON NORTH, M. Am. Soc. C. E.*

Died July 20th, 1911.

The wise men who founded the American Society of Civil Engineers have all passed away.

Their immediate and faithful successors, the men who completed the foundation, and built the superstructure, are in turn fast passing away. The subject of this memoir was one of these.

Edward Payson North was a New Englander by lineage and nativity. He was born at Hartford, Conn., on July 16th, 1835. He died in New York on July 20th, 1911. He was the second son and youngest child of Dr. Milo L. North, a well-known physician, and member of the Class of 1814 at Yale. His mother was Julia Smith, a

* Memoir prepared by Edgar B. Van Winkle, M. Am. Soc. C. E.

graduate of Miss Pierce's famous pioneer school at Litchfield, Conn., her father being the Rev. Daniel Smith, for fifty-three years the pastor of the Congregational Church of Stamford, Conn. Her mother was the daughter of Rev. Cotton Mather Smith, who was for more than fifty years pastor of the Congregational Church of Sharon, Conn. Her uncle, John Cotton Smith, was the last Federal Governor of Connecticut, retiring in 1817.

Dr. North's practice was in Hartford, until about 1838, when he moved to Saratoga Springs, N. Y., and there continued actively at work in his profession until his death some eighteen years later.

As a boy, Edward P. North attended a private school in Saratoga, was taught Greek and Latin by his father, and mathematics by his older brother, Thomas M. North, now a well-known member of the New York Bar. Eventually, he was fitted for college at the Canning School, in Stockbridge, Mass. Having determined to enter the Profession of Engineering, he confirmed his choice and obtained a practical foundation for subsequent professional study, by obtaining two years' service in subordinate positions under the late Oliver H. Lee, first, in 1851, as Rodman on the construction of the second track of the Hudson River Railroad, and subsequently, in 1853, on surveys for and the construction of the Chicago, Alton and St. Louis Railroad.

In 1854 he entered Union College as a sophomore of the Class of 1856 in the Department of Science and Engineering.

Socially he became a member of the Sigma Phi Society, and throughout his life maintained his interest and popularity in that venerable fraternity.

Union College, at that time, had obtained a high reputation for its school of engineering, which had been established and built up by the genius, enthusiasm, and liberality of that gifted scholar, the late Professor William Mitchell Gillespie.

Young North had completed all but the last term of his senior year when he received an invitation to join our late distinguished president, Mr. E. S. Chesbrough, as an Assistant Engineer on the great work of constructing a sewerage system for Chicago. This appointment was too attractive to be neglected. Mr. Chesbrough was a leader in his profession, and the work called for was unique and of the greatest educational value from a professional standpoint. No wonder the young engineer promptly embraced the opportunity offered, leaving his course in college to be completed later. Both Mr. Chesbrough and General Webster, the Sewerage Commissioner of Chicago, have spoken in high terms of the intelligence and reliability shown by Mr. North in this work.

During cessation of activity on this work, in the winters of 1857, 1858, and 1859, he returned to Union College to study mining engineering, and, incidentally, took a post graduate course in chemistry, giving

much time to working on blow-pipe analysis in the chemical laboratory under the immediate supervision of Dr. Charles F. Chandler, since Professor of Chemistry in Columbia University and most prominent as an American chemist.

Mr. North had thus the good fortune to have known in his time the venerable Dr. Eliphalet Nott, then still President of Union College, and to have enjoyed the intimate personal instruction of Professor Gillespie and Dr. Chandler, both not only advanced scientific scholars, but instructors having that rare faculty of systematic clear exposition and thorough teaching, combined with such enthusiasm for their respective professions as to infect likewise those studying under them. It is interesting to note that Dr. Chandler had just begun his brilliant career, and young North heard him deliver his first lecture. Fifty-three years later, Dr. Chandler invited his old pupil to attend his last lecture, and during his address humorously alluded to him as "Exhibit A."

It was while at work in the laboratory in the winter of 1858-9 that Mr. North's eyes began to give way. He continued for another season with Mr. Chesbrough, when it became evident that his eyesight could be saved only by complete rest, and living as much as possible in the open air.

This crisis was happily met. He purchased a small, attractively wooded farm on the banks of Rock River, near Sterling, Illinois, to which he retired with his young wife, having just been married to Miss Kate L. Westcott, of Saratoga Springs, the choice friend of his early youth, who survives him.

The anticipated deprivation from reading and study, and the loneliness of a country life, were by this union transmuted into the sweet pleasures of simple home life. He found congenial employment in the open air in cultivating his farm and garden, while his eyes were constantly spared their use, his wife, acting as reader and amanuensis; so the dreaded years of separation from his profession and kindred proved in reality to be filled with domestic enjoyment and healthful repose. Some four years later, in 1864, his eyesight having become perfectly restored, he once more took up his profession, and received an appointment as Assistant Engineer on the then called Saratoga and Hudson River Railroad, running from Athens to Schenectady, now a part of the West Shore Railroad.

At about this time Mr. North lost his only child, James Westcott North, a boy of six years. In 1866 he was appointed as Chief Engineer of the Stamford and New Canaan Railroad, now known as the New Canaan Branch, in the great system of the New York, New Haven and Hartford Railroad. This road he located and constructed. The location was so excellent that the line he laid down has never to this day been changed in plan or profile. It was on this work that Mr. North experimented with the use of nitro-glycerine in rock excavation. He

must have been one of the first engineers in the country to try it and advocate its use, as its application to blasting had been patented only three or four years before. When we consider its universal present-day use as an explosive, in the shape of dynamite, we can appreciate his far-sightedness as a chemist and engineer. About 1867 he made some preliminary surveys for projected railroads in Iowa, and in 1868 had the good fortune to be connected with the building of that monumental work, the Union Pacific Railway, or the eastern half of the first American transcontinental railway.

General Grenville M. Dodge was the Chief Engineer, Silas Seymour, Consulting Engineer, and Samuel B. Reed, M. Am. Soc. C. E., Chief Engineer of Construction. Mr. North was in charge of the entire work through the Wasatch Mountains, in Utah, some of it of the most difficult and expensive kind, consisting of heavy rock and earth excavation, the protection of the road against the encroachments of a mountain river, tunneling, masonry, bridges, etc., and was very successful in the use of nitro-glycerine in tunnel work. His experience with heavy rock excavation, before dynamite had been invented, was thorough and interesting, and from the data thus obtained, and from previous experience when building the New Canaan Railroad, he subsequently prepared two valuable papers on "Blasting with Nitro-Glycerine"* and "On Blasting Memoranda of Two Blasts Fired April, 1869, on the Union Pacific Railroad."† He assisted in finishing the work in Weber and Echo Cañons, seeing the laying of the last rail and hearing Dr. John Todd's famous prayer on that historic occasion.

In 1872 Mr. North was engaged in making preliminary surveys for a railroad in New Jersey. From 1873 to 1875 he did work for the United States Engineer Corps, as Principal Assistant to Col. F. V. Farquhar, Corps of Engineers, U. S. A., and had charge of the improvement of the upper Mississippi, between the Falls of St. Anthony and St. Cloud. This work Mr. North has described in a valuable paper‡ prepared for this Society. Under the same distinguished engineer he also had supervision of the harbors of Lakes Michigan and Superior, and of the support of the Falls of Saint Anthony at Minneapolis.

In 1876 he was appointed Superintendent of Roads and Streets in the Department of Public Parks of New York City, which department at that time was charged with all public works and their maintenance in the 23d and 24th Wards ("Annexed District"), including all that portion of the city above the Harlem River, some 13 000 acres. This was a suburban district with only one paved street; the other streets and roads were to a degree macadamized, but the large preponderance of them were country roads, merely surfaced with earth.

* *Transactions*, Am. Soc. C. E., Vol I, p. 13.

† *Transactions*, Am. Soc. C. E., Vol. I, p. 214.

‡ *Transactions*, Am. Soc. C. E., Vol. VI, p. 268.

With meager appropriations, Mr. North made a great improvement in the condition of the streets and roadways. Discarding haphazard primitive methods, he introduced scientific and systematic treatment, and made several practical experiments of much value.

In 1878 Mr. North, in conjunction with John Bogart, M. Am. Soc. C. E., and the late George S. Morison, Past-President, Am. Soc. C. E., prepared an exhibit to represent the American Society of Civil Engineers at the "Exposition Universelle," held in Paris in 1878. This exhibit consisted mostly of designs and photographs of great engineering works recently constructed, or in course of construction, in the United States—notably such as foundations, dams, locks, bridges and viaducts, hydraulic machinery, railroads and their equipment, river and harbor improvements, etc.* So interesting and well prepared was this exhibit that the expert jury reporting on it asked that an exceptional prize be awarded.

Mr. North visited this exhibit after its installation in Paris, passing the summer of 1878 abroad, where he made special study of European roads and their care, particularly as to the use of asphalt for street surfacing in London and Paris. The result of this study was the preparation of a very interesting and exhaustive paper entitled "The Construction and Maintenance of Roads."† This treatise was esteemed of such originality and value as to become a standard, and won for him, in 1879, the Norman Medal of the Society in special commendation for its merit as a contribution to engineering science.

In 1880 Mr. North was employed as Consulting Engineer in relation to irrigation for a sugar plantation near Santiago, Cuba, and afterward relative to a water supply for the Ortiz Mine, in New Mexico.

He became, in 1881, Chief Engineer and General Superintendent of the Sinaloa and Durango Railroad, with headquarters at Culiacan, California. He made the first reconnaissance for it, including crossing the Sierra Madre, at an elevation of 10 000 to 11 000 ft. above sea-level, and superintended construction until the rails were partly laid on the only section ever built, namely, from Altata, on the Gulf of Mexico, to Culiacan, the capital of the State of Sinaloa, some 26 miles. The sanitary conditions of Culiacan were deplorable, and the natives so careless about vaccination, that varioloid was prevalent, and was contracted by Mr. North, forcing him eventually to return to New York, where he became the Vice-President and General Manager of the Railroad Company.

Subsequently he acted as Consulting Engineer for the electric subways in New York, and from time to time was called as an engineering expert to various cities, one of which, in 1906, was Joliet, Ill., to examine the water-power development under way by the Sanitary District of Chicago.

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* *Transactions*, Am. Soc. C. E., Vol. VII, p. 317.

† *Transactions*, Am. Soc. C. E., Vol. VIII, p. 95.

Mr. North visited Europe a number of times. He attended the Fifth International Congress on Inland Navigation, held in Paris in 1892, and presented a report, entitled "Notes on the Relations between Railroads and Waterways in the United States," which had the honor to be printed by the Congress in English, French, and German. He also took a prominent part in the discussion at that Congress. He studied the system of canals in Holland, the control of rivers in France, and, when in Italy attending a session of this Inland Navigation Congress—during his last visit to Europe—at Rome he made a special study of the railways just taken from the contractors by the Government, and prepared an interesting article on Italian railways.* He made many friends among distinguished European engineers and officials, resulting in much kindness and marked courtesies on many occasions, particularly in Rome and when visiting Sicily.

Mr. North, from the reputation established by his paper, "The Construction and Maintenance of Roads," became the leading authority at that time on roadway pavements, and in 1895, under Mayor Strong's administration, was appointed "Water Purveyor," actually the head of that Bureau in the Department of Public Works of New York City, charged with paving and repairing streets. Under his jurisdiction at this time was initiated a general change from stone to asphalt pavements, many miles of the latter being laid and the appearance of the City noticeably improved thereby. A minor work, but one of great practical utility, which he carried out at this time in connection with laying asphalt pavements for the roadways of Park Avenue, was to modify the absurd and dangerous crowning of its pavement, lowering the crown of the roadway as much as 18 in. in some places. While Water Purveyor he read before this Society a paper entitled "The Influence of Rails on Street Pavements,"† not only valuable in itself but in the information brought out in the discussion which followed its publication.

Two years later, in 1897, he was made Consulting Engineer of the Department of Public Works of the City of New York, a position of great importance in view of the immense amount and variety of work carried on by that Department, and the new problems in municipal engineering constantly arising.

In 1897 Mr. North was appointed Consulting Engineer to Governor Black's Canal Investigating Commission, and in association with Lyman E. Cooley, M. Am. Soc. C. E., made a report in 1898 embodying certain modifications in the proposed route for the enlargement of the canals of the State of New York, and very full estimates of the cost of construction of the enlarged canal from Lake Erie to the Hudson River.

From a literary standpoint, Mr. North wrote well and clearly on

* Published in *The Railroad Gazette.*

† *Transactions*, Am. Soc. C. E., Vol. XXXVII, p. 70.

many professional subjects. Whatever he wrote, whether formal papers, journalistic contributions, or technical reports, was characterized by earnestness, thorough study, and sincerity.

In addition to the papers prepared by him for this Society, and for other professional bodies briefly mentioned above, he was at one time a frequent contributor to both *Engineering News* and *The Engineering Record*. He also contributed valuable articles on transportation to *The North American Review* and to *The Forum*.

One of his last papers was prepared for the International Engineering Congress of 1904, on "The Concurrent Development of Traffic on Improved Waterways and on Railroads."*

Numerous additions were made by Mr. North to the discussions following the reading of papers before this Society. His technical education, the broad field of his practical experience, and his great store of well-garnered facts, gleaned from extensive professional reading, was an admirable equipment for taking a prominent, or leading part, in such discussions.

In this connection might be recalled his strength as a debater. Speaking of his earnest advocacy of the tariff and its operations, it was said of Mr. North, by one who knew him well: "Nobody ever entered into a controversy with him on the subject more than once, for his wide range of information, and wonderful memory for statistics, made him an ideal debater."

The great diversity of his interests and knowledge can best be appreciated by glancing at the *Transactions* of this Society and noting his contributions to the various discussions recorded: "Resistances of Railroad Curves," "Nomenclature of Building Stones and of Stone Masonry," "Railroad Construction," "Inter-oceanic Canal Projects," "Water-proof Coverings," "Temperature of Water at Various Depths in Lakes and Oceans," "Enlargement of the Erie Canal," "Preservation of Timber," "Preservation of Forests," "South Pass Jetties," "English and American Railroads Compared," "Railroad Levels," "Sewage Disposal," "Mean Horse-Power of a Stream," "Street Railway Track," "Subaqueous Foundations," "Use of Asphaltum in Building Sea Walls," "Construction of Railway Tracks," "Right of Way for Railroads," "The Proposed Lake Erie and Ohio River Canal," "Brick Manufacture and Brick Pavement," "The Holland Dikes," "Controverted Questions in Road Construction," "Electric Rock Blasting," "Inland Transportation," "The Water-Works of Denver, Colorado," "Effect of Depth upon Artificial Waterways," "Construction of a Water System for Placer Mining and Suggestions for a New Method of Dam Building," "Asphalt and Asphalt Pavements," "Lake Front Improvements, Chicago, Ill.," "Theory and Practice of Special Assessments," "Economic Depth for Canals," "Road Building," "Street Grades and Cross-Sections,"

* *Transactions*, Am. Soc. C. E., Vol. LIV, Part B, p. 475.

"Jordan Level, Erie Canal," "Canals from the Lakes to New York," "Railroad Freight Differentials," "The Bohio Dam," "Improvement of Rivers," "Nicaragua Canal," "Preservation of Materials of Construction," "Pavements," and "Forests, Reservoirs, and Stream Flow."

Mr. North might almost be called one of the founders of the American Society of Civil Engineers, although not actually one of those thirteen far-seeing men who in 1852 banded together, not to gain more pay, do less work, or stifle competition by limiting the number of young men entering the profession, but rather, in union, to broaden its usefulness by increased education and higher ethics, and to increase that mutual respect which inevitably follows better personal acquaintance between men of honor and intellect.

Mr. North was not actually one of these founders, yet he was in fact a founder inspired with all their traditions, for he was one of the earliest to be chosen to reinforce the founders, the majority of whom, far called by the exigencies of war, had been unable to maintain an active organization after 1854.

With a restored Union came the revival of our Society in 1867 and the new inspiration inherent with younger men. Mr. North was one of the first of those chosen at this time, his membership dating from December 4th, 1867. His sponsors were such men as E. S. Chesbrough, General George S. Greene, Alfred Craven, and others.

He was ever an enthusiastic and loyal member, rarely missing any of its meetings, always ready to serve by actual work and counsel to aid in its firm establishment and uplift. Mr. North joined the Society when it had but 26 members. When he died there were not less than 6 000 on its rolls. Our Society has attained grand proportions and prestige. The unselfish men like Edward P. North, who alone for the love of their chosen profession laid its solid foundation, were unconscious that they were preparing for a grand superstructure that would be their lasting monument.

He was made a Director of the Society in 1891, and was chosen its Vice-President for 1898 and 1899. He always attended the Annual Conventions and added not a little to their attractiveness by his genial presence and ability to discuss the papers there presented.

An unusual and dominating factor in the professional career of Mr. North was his love for his profession first, and always for itself, and only secondarily from a business standpoint. This was largely due to his intellectual nature, and that he had become imbued with the enthusiasm of his great teacher, Professor Gillespie, for the theoretical beauties of his profession. With neither master nor pupil would mere remuneration weigh against work intrinsically interesting.

No memoir of Mr. North would be complete without reference to his character and characteristics. He had a pleasing personality, well suited to his social disposition, and naturally gained and kept many friends.

Perhaps his most striking quality, at least the one most appealing to the younger members of the Profession, was his cheerfulness and affability of manner. He was deservedly popular with them, as he was ever ready to give them help and encouragement, not only by word but he would often go much out of his way to aid young engineers by giving, or securing for them appointments. One who was with him as a young assistant in Mexico thus speaks of him:

"He was an extremely hard worker, and expected hard and plenty of work from his assistants, but was very much respected and beloved by all of them. I always retained an affectionate and filial regard for him and maintained with him most delightful relationship up to the time of his death. He always looked upon and spoke of those who worked under him as 'his boys.'"

He was entertaining in conversation, and had the delightful faculty of telling of his wide professional experience and incidental adventures, picturesquely, while yet quite free from boast or egotism.

His words were always so tempered with humor and kindliness that no adversary in debate ever felt wounded by them, even when they were most critical. He had that faculty, often so rare with those ardent in debate, of arming himself with logic and facts alone, and never being tempted to use ridicule or sarcasm.

In his professional work he was most honorable, honest, and independent. Of undoubted integrity, no prospective personal advantage ever warped his judgment or action. His superiors, under whom he worked, all spoke of him as thoroughly reliable and as an accomplished man of excellent ability.

In a word, Edward P. North had intellect, manliness, and gentleness, "That gentleness, which, when it mates with manhood, makes a man."

LA FAYETTE OLNEY, M. Am. Soc. C. E.*

DIED MARCH 2D, 1912.

La Fayette Olney, the son of James and Phœbe Smith Olney, was born on June 20th, 1836, at Westmoreland, N. Y., and died on March 2d, 1912, at Mahwah, N. J.

He entered Union College, Schenectady, N. Y., on April 23d, 1857, at the age of twenty-one, taking the scientific course, and was graduated with the degree of A. B.

For many years he was a member of the firm of Wells and Olney, Architects, at Lawrence, Kans., and was also employed extensively in surveys of Western railroads, particularly those now forming part of the 'Frisco and Missouri, Kansas and Texas Railroad systems.

* Memoir prepared by the Secretary from information furnished by Mrs. Olney, and from papers on file at the House of the Society.

He made extensive surveys in New York City, and was also connected with the New York State Department of Public Works in the office of the State Engineer and Surveyor, having charge of work on New York State canals at Owego, N. Y. Later, he opened an office at 99 Nassau Street, New York City, where he was engaged as Consulting Engineer until his health failed and compelled him to give up active business.

Mr. Olney was a great lover of nature, as well as of the artistic, and had much ability as an artist, doing a great deal of beautiful wood carving, and painting and sketching in oil and water colors. During the later years of his life he copied many of the old masters in the galleries of France and elsewhere. He was a great reader and a fluent French, German, and English scholar. He travelled extensively, crossing the Atlantic fourteen times.

Mr. Olney did much good and charitable work and aided many, both in his own family and outside of it. His was an admirable character, strong, courteous, and courageous. He was a Mason and a Knight Templar. His widow, Elizabeth Hopper Hopkins Olney, formerly of Glen Cove, Long Island, survives him.

Mr. Olney was elected a Member of the American Society of Civil Engineers on October 7th, 1868.

GEORGE HOWARD WHITE, M. Am. Soc. C. E.*

Died December 29th, 1911.

George Howard White was born at Grafton, Mass., on June 9th, 1851. After a course in surveying and higher mathematics at the Grafton High School, he, in January, 1870, entered the office of Mr. W. P. Granger, a civil engineer, at Worcester, Mass. He remained with Mr. Granger until April 1st, 1870, when he was appointed Rodman on the surveys for the Adirondack Railroad, being promoted, within two months, to the position of Levelman. He remained on this work until February, 1871.

In the spring of the same year, Mr. White entered the Worcester Polytechnic Institute. After passing all the examinations except those of the senior year, he, in 1872, entered the office of Mr. John Ellis, at Woonsocket, R. I., with whom he remained until January 1st, 1873. He was also employed by Mr. Herbert Keith on surveys of preliminary lines in the vicinity of Boston, Mass.

In April, 1873, Mr. White went West, settling in Minneapolis, Minn., where he was engaged on city and county surveys and on road

* Memoir prepared by the Secretary from information on file at the Society House and from a memoir prepared for the Engineers' Club of Minneapolis, Minn., by its Secretary, J. G. Anderson, Assoc. M. Am. Soc. C. E.

construction for Mr. Franklin Cook, the owner of some stone quarries along the Mississippi River, in the eastern part of the City. In 1874, he was appointed Assistant to the Chief of Party on the survey of the St. Croix River, from Taylor's Falls to its junction with the Mississippi. After the completion of this work, Mr. White returned to Minneapolis in the employ of Mr. Cook, remaining until March, 1875, when he was appointed Instructor in charge of field practice and mapping, in the Civil Engineering classes at the Worcester Polytechnic Institute. He held this position until the fall of 1876, when he was graduated, as a Civil Engineer, with the Class of 1876 of the Institute.

Mr. White then returned to Minneapolis and, in August, 1877, was appointed Assistant Engineer, under the direction of the Assistant General Superintendent, on the Chicago, Milwaukee and St. Paul Railway, with headquarters at Minneapolis. It was at this time, and under his direction, that the Company's engineering office in Minneapolis was established. Mr. White had charge of the construction of what is known as the "Short Line" between Minneapolis and St. Paul, which was built by the Chicago, Milwaukee and St. Paul, and of the erection of the "Short Line Bridge" over the Mississippi in 1878 and 1879. He was also in charge, for the Railway Company, of the construction of the foundations and masonry of a new elevator, as well as of new shops at South Minneapolis. Later, he was made Engineer in Charge of the Division Office at Minneapolis, and of surveys relating to maintenance of way on various Western Divisions of the road.

In January, 1884, Mr. White was appointed Professor of Civil Engineering at the Worcester Polytechnic Institute, and while in this position, organized and built up the Civil Engineering Department. He made a special study of bridge engineering, and was engaged in private practice as a Consulting Bridge Engineer, in addition to his educational work.

In June, 1901, he was made Assistant Superintendent of Sewers at Worcester, Mass., and held this position until January, 1906, when he resigned to return to the Chicago, Milwaukee and St. Paul Railway Company, with headquarters in the Division Engineer's office at Minneapolis, Minn. The Company's "Transcontinental Line" (the Chicago, Milwaukee and Puget Sound Railway), from the Missouri River to the Pacific, had been commenced, and the location and construction of its eastern end was done under the direction of the Division Engineer's Office at Minneapolis.

After the completion of this work, Mr. White made a physical valuation of the Chicago, Milwaukee and St. Paul Railway Company's property in Minnesota for the use of the Minnesota Railroad and Warehouse Commission in the "Rate Hearing Case." He was engaged

on this work up to within a few months of his death, which occurred at Minneapolis, on December 29th, 1911.

Mr. White was an earnest student of engineering and was honest and conscientious in all his dealings, his career having been marked by extreme devotion to duty both in public and private life. During his residence in Worcester, Mass., he was prominent in public school affairs, being, for nine years, a member of the School Board; he was also a Deacon of the Free Will Baptist Church. He was one of the charter members of the Engineers' Club of Minneapolis, and was elected an Honorary Member of the same Association on February 8th, 1884.

He is survived by his wife, three sons, three daughters, and five grandchildren.

Mr. White was elected a Member of the American Society of Civil Engineers on May 2d, 1883.

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PAPERS AND DISCUSSIONS

STATE AND NATIONAL WATER LAWS, WITH DETAILED STATEMENT OF THE OREGON SYSTEM OF WATER TITLES.

By John H. Lewis, Assoc. M. Am. Soc. C. E.
To be Presented November 6th, 1912.

In all irrigation, power, and other developments involving the diversion and use of water, much attention is given by the engineer to a detailed study of the available water supply. Whether or not legal title to the necessary water can be secured, is a matter usually left by the engineer for the consideration of the project's legal adviser. The lawyer, as a rule, is unfamiliar with the units for measuring water, knows nothing as to the effects on the stream of diversions or of return seepage, and can make but little use of elaborate hydrographs and water supply data prepared by the engineer. He confines his studies strictly to the legal phases of the problem as he understands them, assuming that the engineer will attend to what he considers the technical part of the work. The inevitable result is that the promoter, who usually understands only the financial end of the project and is relying on his engineer and legal adviser to do their part, suddenly awakens to the fact that he has expended a large amount of money on a project where the legal title to a part or all of the water supply is in some one else.

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

In the seven years' experience of the writer as State Engineer of Oregon, many disasters have been observed where the primary cause was clearly a failure on the part of the promoter, or the engineer, to appreciate fully at the outset the importance of investigating thoroughly those engineering questions which are semi-legal in nature. The fault usually lies with the engineer in making a water supply report which purports to be complete, but does not treat of the legal relation between the proposed diversion and all other diversions and rights along the entire stream. The engineer is much better fitted by training and experience to cover this disputed territory than the lawyer.

In many States, however, it is absolutely impossible for any one to secure definite information as to water rights or titles. The water laws of such States cannot be found by a study of the statutes alone, but must be sought for in a long series of decisions by the State Supreme Court. These decisions are based more on the logic of judges than the statutes of the Legislature, and are usually so conflicting that able lawyers differ in opinion as to the most fundamental points. In such States, it is the duty of the Engineering Profession to lead in the movement for the enactment of better water laws. It is impossible for the lawyer or statesman to frame proper laws without the co-operation and assistance of the engineer, for he alone knows the difficulties to be overcome in undertaking a new enterprise; and he, of all classes, should know what legislation is needed to protect all interests engaged in such an enterprise. The legal profession has great reverence for precedent, and, besides, there are many lawyers who feel that a reform in the water laws would affect business seriously, by reducing the amount of litigation. It is said on good authority that the cost of litigation over water rights in some of the Western States has exceeded the original cost of the works. The recent collapse of the irrigation bond market is evidence of the fact that capital has awakened to the importance of definite water laws, so that a reliable abstract of water title can be prepared prior to construction. Therefore, looking at this matter only from the selfish standpoint, it is the duty of the engineer to lead in the reform of State and National water laws, so that the confidence of capital may be speedily restored and that development may proceed. The best lawyers in every community will always be found on the side of

reform, for they realize that their prosperity depends on the general prosperity of their community and State.

The wonderful progress, made during the past decade, in the reclamation of arid lands, through the agency of the Carey Act, and the United States Reclamation Act, has served to educate the public as to the value of water and the importance of some interstate or National laws relating to diversions from interstate streams, or their tributaries. Formerly, hundreds of small diversions were necessary before any appreciable effects on a large interstate stream were noticeable in the lower State, but now, in some cases, the entire flow of a great river may be stored for several years in a reservoir and the flow in the adjoining State is affected so suddenly as to be a shock and a menace to its citizens. Such enormous developments were not contemplated at the time of the framing of our State and National constitutions, and some new legislation must be framed and enacted to cover these contingencies, even if it necessitates the stretching or amending of these constitutions.

Some interstate legislation is absolutely essential if the highest development of our streams is to be accomplished. Each river, from its head-waters to its mouth, should be treated as a unit, regardless of State lines. For instance, it should be possible to store water in Colorado for use in Southern California, the stored water being protected by Federal or interstate law as it passes through three intervening States. Again, the construction of a power-plant, to utilize the entire low-water flow of a stream in one State, may interfere seriously with subsequent diversions for irrigation in the adjoining State immediately above such plant. A number of cases of litigation between States over a proper diversion of interstate streams have already been carried to the Supreme Court of the United States, and some definite action should be taken by Congress before these decisions become hopelessly confused, as is the case in most States where no statutory laws have been available for the guidance of the Courts. The National engineering societies should lead in the discussion of this important subject, and assist in the framing of necessary interstate water legislation.

Just as we have done away with county control of waters, for State control in Oregon, so must we supplement State control by some system of interstate or Federal control for interstate streams,

and the same system which has been found satisfactory for the State should be extended to interstate waters.

The water laws of Oregon, though not specifically referred to, have been endorsed as a model for other States by resolutions of the National Irrigation Congress. It required a four-year campaign, supplemented by the powerful club, the power of direct legislation through the initiative, now in the hands of the people of Oregon, to force this law through the legislature. It has now been in successful operation for three years, and the results accomplished have been far in advance of what was hoped for, even by the most sanguine of its supporters.

For the foregoing reasons it is believed that a brief statement of the fundamental principles underlying the Oregon system of water titles, together with the results thus far accomplished, will be of interest to the Engineering Profession generally, and may assist in the solution of some of the conservation and water-power problems which are now calling for the highest degree of statesmanship on the part of our representatives in Congress.

The value of this paper will be greatly extended if it receives careful consideration and full discussion by engineers in the Eastern, Southern, and Central States, as they look at these questions from an entirely different view-point. The general impression seems to prevail that a law suitable for the Western arid States would not be adapted to conditions in the Eastern humid States. It is high time that this notion be dispelled; for the diversion or storage of 100 sec-ft. of water from a stream for irrigation or domestic use in Oregon would have the same effect on lower appropriators as a like diversion for an eastern stream for any similar uses to which water may be put in such section. Until recently, the common law doctrine of riparian rights, in all its rigors, applied to conditions in the western or humid portion of Oregon, while the doctrine of appropriation for beneficial use prevailed in the eastern or arid portion of the State. The same law has now been made applicable to all parts of the State.

The writer will now endeavor to show that this same system of water titles, which is essential to development in the arid States, is equally applicable to Eastern and Southern States, so that this discussion may be of interest and perhaps of value to engineers in all parts of the country.

"The United States inherited from Europe two great systems of jurisprudence, one the Common Law, and the other the Civil Law. The eastern part of the territory of the United States began with the Common Law. The territory west of the Mississippi inherited the Civil Law, through the prior sovereignty of either Spain or France, or both. The Common Law has invaded most of the area of the Western States, formerly subject to the Civil Law, yet there are many great questions of both title and right still dependent there for settlement upon the rules of the Civil Law. Among the great questions thus dependent is the law of waters."

"The basis of the Civil Law which comes to any State of the American Union, is the law of imperial Rome. It will surprise any one who reads the translation of the Roman water law to see how perfectly and fully they understood the whole question. They had worked out every problem, and we may safely say that they understood the subject as well as we do to-day, and no one can say that the old Romans did not bring to their law of water a common sense and equity not exceeded by our courts to-day."*

The modern water laws adopted by a few of the Western States, including Oregon, are based on the Roman Law. The basis is priority of appropriation and beneficial use. The Oregon territory, however, was acquired by discovery, and therefore this State started off with the same system of jurisprudence—the Common Law—as the eastern part of the United States. The early decisions of our Courts were in harmony with decisions in the Eastern States, yet these decisions have been almost completely reversed in a State where occurs, perhaps, the heaviest rainfall (in Tillamook County, 139 in.) of any point in the United States. The writer, therefore, cannot see any reason why a law similar to that of Oregon cannot be adopted and upheld in any other State of the Union where it is necessary to divert water.

Our system of land titles is well understood in all States. We know the value of an abstract of title. Why should we not have a definite system of water titles in all States, and why not insist on just as definite showing of water title before the purchase of a water project, or before the investment of money in a new enterprise? Entire streams are now being diverted or stored in the Eastern States for municipal supplies, for supplying large canals, for the development of power, etc. In many cases, water is carried from

* Ware, on Roman Water Law.

one stream into another and from one State into another. Soon the State and Nation must join in the storage of water for the control of floods and to aid navigation. This water, in passing down the stream, will benefit many private power projects, and these should be compelled to contribute to the cost in proportion to the benefits received. It is the writer's belief that definite State and National laws relating to water would be of as great benefit to the Eastern as to the Western States.

Riparian Rights *versus* Appropriations.

Before going farther into this subject, it might be well to explain briefly the distinction between the common law doctrine of riparian rights, and the civil law doctrine of appropriation and use.

The common law doctrine of riparian rights originated in England, where it was not necessary to divert and use great rivers for the domestic supply of cities, where no extensive diversions were necessary for the supply of canals, or for water-power projects, as we know them to-day, and where irrigation was not necessary. It was more a question of drainage than of irrigation.

"Riparian rights" are such as follow or are connected with the ownership of the banks of streams or rivers.

"Every proprietor of land on the bank of a river has an equal right to the use of the water which flows in the stream adjacent to his lands, as it was wont to flow, without diminution, pollution or alteration, unless his right has been limited by grant, license or prescription. No proprietor along the bank of a stream, however, has a right to use the water to the prejudice of other proprietors above or below him, unless his right has been enlarged by grant, license or prescription. He has no property in the water itself, but a simple usufruct while it passes along. * * * Though he may use the water while it runs over his land, he cannot unreasonably detain it, nor give it another direction, and he must return it to its ordinary channel when it leaves his estate. These rights are inseparably annexed to the soil, and pass with it, not as an easement or appurtenance, but as a part and parcel of the land. This property right can be regarded only as a corporeal hereditament, belonging to and incident to the soil, the same as though it were stones or grass. The right does not depend on appropriation, and is not suspended or lost by nonuser, but may be lost by grant or prescription, or long adverse use; or in the semi-arid states, by condemnation."*

* Mills' Irrigation Manual, p. 18.

The doctrine of appropriation for beneficial use, as far as the United States is concerned, originated in California and other Western States, out of the necessities of the miners and early settlers. It is based on the familiar maxim that "he who is prior in time is prior in right." Water is treated as a wild thing which becomes the property of him who captures it first. It can only be retained by him so long as he puts it to beneficial use. If the supply is not sufficient for all, the ditch of the last appropriator must be closed to protect the prior appropriator. This system of titles is simple, easy of enforcement, and is the only system which is adapted to the needs of the people in this age of big developments.

These two systems are in direct conflict, and cannot both be applied on the same stream at the same time, for streams cannot flow undiminished to the ocean, and at the same time permit diversions of water for beneficial use.

"From the history of this subject, dating * * * from the earliest period of Egyptian history, down to the period when extensive appropriations were made for mining and agriculture in the arid and semi-arid West, we are unable to find any controversy between those claiming as riparian proprietors and those engaged in diverting and conducting the water to non-riparian lands. It seems to have been an accepted fact that the water was the property of the public, and when the necessities of the people required that it should be conducted from the stream and applied to the soil for the production of crops, the right to do so was unquestioned."*

Early Laws.

Oregon was admitted as a State on February 14th, 1859. Its constitution contains no provisions relating to water. Many decisions had been rendered by our Supreme Court before the adoption by the legislature of the first water laws, in 1891. These laws were in harmony with the customs of the miners, and provided for the posting of a notice on the bank of the stream, setting forth the extent and purpose of the claims, and the recording of a copy of such notice with the county clerk. Work was to begin within six months, and no time limit was fixed for completion. There was no supervision by the county or State. As a result, each applicant usually claimed many times the water needed, and usually all the water in the stream at

* Mills' Irrigation Manual, p. 6.

that point. The records, therefore, were of no public value, and in one case a trip of 1 000 miles would have had to be made to consult all the records relating to a single stream. Litigation was constantly in progress on some streams. The decrees settled only the rights of the parties interested, and were not final because they did not involve all parties on the stream, including the State, which is vitally interested in the unappropriated waters, as trustee for future users. This litigation impoverished the water users, discouraged settlement and investments, and promoted community quarrels which threatened the peace and safety of the citizens. The term "water right" became in practice synonymous with litigation. To overcome these difficulties, a complete system of State control was provided in February, 1909, and is based on the police power of the State to preserve the peace and safety of its citizens.

The New Law.

All water within the State of Oregon was declared by law to be the property of the public. This declaration merely states the law as it exists to-day in every State of the Union. It is the purpose of this new law to regulate the use of the State's property in the interest of the public, without injury to prior vested rights. This regulation is possible under the police power of the State.

A board, of which the State Engineer is chairman, was created to have charge of this property.

It is the duty of this board:

A.—To determine and record all rights to the use of this public property which had become vested prior to the adoption of the law;

B.—To grant rights, for beneficial use in the State's unappropriated waters, to those who make proper application therefor;

C.—To protect recorded rights to the use of public waters by regulating diversions from streams.

The law and its operation will be discussed under these three heads, taking them up in order.

Determination of Old Rights.

A complete and reliable record of all old rights is the foundation for their protection as well as the basis for the State's administrative

system. Such a record is essential, in order that the public may know the extent of its unappropriated waters. As streams do not follow county lines, or begin and end in particular judicial districts, it was found absolutely necessary to create a special tribunal to ascertain and record these old rights. It is called the Board of Control, and is composed of the State Engineer, and the Superintendents of each of the two sections into which the State is divided.

The Board of Control is an administrative body having only subordinate judicial powers. While it has much the same powers as a Court, and follows, where practicable, the usual legal procedure, yet it is free to depart from such procedure where necessary to facilitate action. It operates in much the same manner as the Railway Commission, or the Board of Health. It knows what information is necessary for a proper determination of early rights. It goes after this information, and does not accumulate thousands of pages of useless and conflicting evidence. Its success has been due largely to its freedom in this respect. Any water user can submit his claim to the Board, if he so desires, without the necessity of employing an attorney. The Board assists him by preparing blank forms* and by making surveys and maps, so as to minimize his expense. In case of contests, the Board pays for the taking of additional evidence.

The determinations are not only inexpensive, but speedy and effective, as shown by the record. The members of this Board are employed by the people of the State for this particular work. It is to their interest, as well as that of the water user, that these determinations be made speedily, and that the determinations are effective.

The first step in the adjudication of water rights on any stream is to file a petition with the Board, signed by one or more water users, requesting that a determination be made. If conditions justify such action, and funds permit, the State Engineer is directed to make a survey as a basis for such determination. Maps are prepared showing each ditch, the location and extent of land irrigated, and such other information as may be necessary in defining the right.

Next, a form containing questions is sent by the Superin-

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* A full set of the forms and instructions used in Oregon was furnished by the Author, but as they are too voluminous to be published here, they have been filed in the Library of the Society, where they may be examined by those especially interested.

tendent to each person or company claiming a water right from such stream. Each claimant is required, under penalty of forfeiture, to fill in the information necessary for a proper determination of his right. The Superintendent, at the time and place fixed in the notice, proceeds to collect these statements, which are certified under oath before him. The maps prepared by the State Engineer are at such time submitted to inspection by the water users, who must accept or reject them. If not accepted, maps must be prepared and submitted by the claimant showing the information as he thinks it should be. Certain fees are payable at this time.

After all claims have been filed on these forms, a second notice is sent by registered mail to each water user, fixing a time and place when these claims will be submitted to the inspection of all interested parties. Within 5 days after the closing of this period of inspection, any claimant may contest the claim of another, each being required to pay $5 for each day required in the taking of testimony in such contest. The deposit of the winning party is returned to him. In this way, the determination of early rights is practically left to the water users themselves, and trivial contests are discouraged. The new comer will contest vigorously any exaggerated or untruthful statement by any prior appropriator. The more these early excessive appropriations are cut down, the more valuable is his own right, and the more likelihood of his securing water. The early appropriator, knowing that his claim will be submitted to the inspection of his neighbors having subsequent rights, is usually modest in his claim.

The original evidence, with that taken in contest cases, is then submitted to the Board of Control, and, as soon as practicable, an order determining the relative rights of all parties is entered. This order becomes effective immediately, and can be enforced by the appointment of a water master.

The order, together with all testimony collected by the board, is filed later with the Circuit Court of the proper county for confirmation. If no exceptions or appeals from the Board's determination are filed with the Court, an order is entered confirming the same. Otherwise, the decree remains in force until the exceptions or appeals are tried out, and a modified decree is entered. Appeals from the decree of the Circuit to the Supreme Court are provided for, if

taken within six months from the entry of the decree in the Lower Court.

On confirmation by the Court, and after the period for appeal has expired, a water right certificate is issued by the Board of Control to each owner of a right.

There is considerable demand for making the decisions, final with the Board, subject to appeal to the Courts, in the usual manner. Such procedure would greatly simplify matters, and hasten materially the issuance of certificates. It would be clearly constitutional, in the writer's opinion, as indicated by decisions in Wyoming and Nebraska.

Water Right Certificate.

The water right certificate is record evidence of the holder's right to water. It is to his water title what a patent from the United States is to his land title. Thereafter his title to water for irrigation passes with the land on which it is used. An abstract of land transfers will then serve to show chain of title to both land and water.

This certificate, which is issued to the claimant by the Board after final determination of his right, embodies all the fundamental principles of a water right. They are few and simple, and, if kept clearly in mind, will give a good understanding of the water laws of Oregon as well as of a number of the Western States.

These fundamental principles are as follows:

$$\text{Beneficial Use:}\left\{\begin{array}{l}\text{Priority,}\\ \text{Purpose,}\\ \text{Period,}\\ \text{Place, and}\\ \text{Quantity.}\end{array}\right.$$

Beneficial use is the great underlying principle. Because of this principle, there are no water monopolies in Oregon. The barter and sale of water, apart from the use, and the holding of water without use, for speculative purposes, are unknown. Our water resources, greater perhaps than those of any other State in the Union, are still largely the property of the public, and are offered practically without price to him who will put them to use.

Priority.—The priority is expressed by a date. It is the most important element in considering the value of a water right. It is the date when the present owner or his predecessor first diverted

water or took the first legal steps to initiate his right. A ditch may have several priorities, if the original appropriation or intent has been enlarged at a later date.

Purpose.—The purpose, whether for irrigation, power, mining, manufacturing, or domestic use, is an important element of the title. A right acquired for one purpose should not be transferred to another without loss of priority. A change from power, which does not consume water, to that of irrigation, which consumes a large part of the water, will affect lower rights seriously. Such changes are against public policy. The user has no property right in the water, only a right for a particular use.

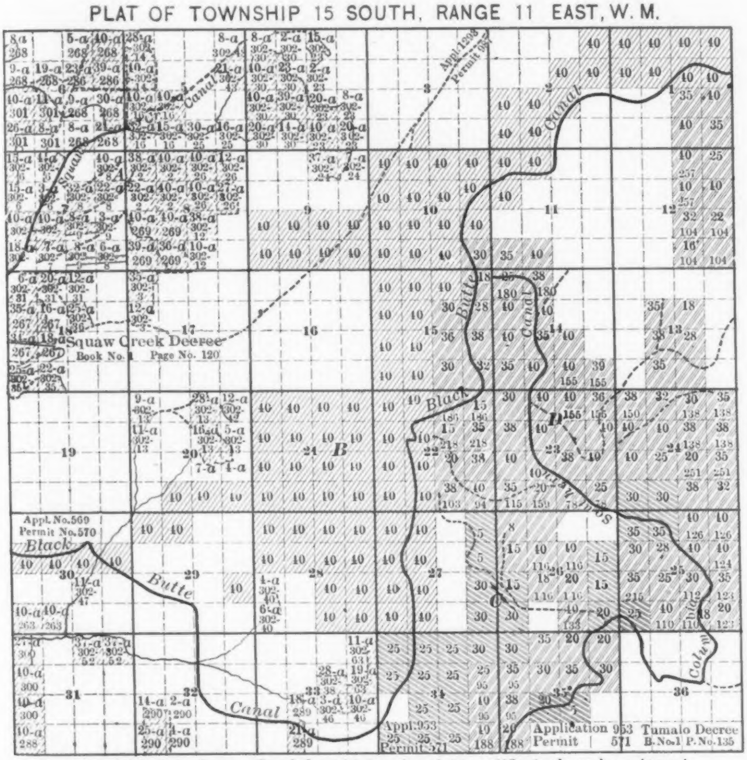
Period.—The period of use is important. A right for mining during the winter months cannot be extended to include summer use, as this would conflict with perhaps prior rights for irrigation. A right for spring irrigation of grain, and long used for such purpose, cannot be enlarged to include fall irrigation of alfalfa, without injury to others. The increased use at a different period would damage vested rights below.

Place.—The place of use is fundamental to a proper definition of the right. Water cannot be branded or fenced like cattle to show private ownership. The nearest approach to such system is to tie it to the place of use. Therefore all water should be made appurtenant to the place of use, otherwise there can be no stability of water titles, and the public records at Salem will be of little use. (See Fig. 1.)

No transfers should be permitted without loss of priority, except in the most extreme cases, and then only after proceedings and record before the Board of Control. The return water from prior appropriations is counted upon by subsequent appropriators as a basis for their investment. Even a relatively small displacement will affect the return seepage, as the laws of underground water are uncertain. There could be no stability of water titles if the law permitted indiscriminate transfers in the place of use. If the use terminates, the right should terminate. Even if a transfer is made after notice to all parties, it is unfair to the lower appropriator, as anticipated injury can seldom be proven. The public gains nothing, as new appropriators are always eager to put such water to new uses under a subsequent priority.

To permit a change in character of use, or change in the place of use, therefore, would serve to make water titles unstable, and open the way to the barter and sale of water, which is in direct conflict with the doctrine of beneficial use, on which the entire system rests. Unless water titles can be made definite and certain, capital cannot be expected to invest in new enterprises.

PLAT OF TOWNSHIP 15 SOUTH, RANGE 11 EAST, W. M.



A—Decreed rights from Squaw Creek for which water right certificates have been issued.
B—To be watered under permit from Blue Lake and Metolius River.
C—To be watered under permit from Crater Creek and storage in Tumalo Reservoir.
D—Incomplete rights under Tumalo Creek adjudication; project not yet completed, including storage in Tumalo Reservoir.

FIG. 1.

Quantity.—The maximum quantity of water granted in the certificate is an important qualification to the general limitation of beneficial use. This is expressed in cubic feet per second, which is the standard unit for rate of flow. Water for storage purposes is ex-

pressed in acre-feet, which is the unit of volume. If a greater quantity than allowed is desired, this can be delivered by the water master under a rotation system, that is, where two or more combine, each using the total allotment, part of the time. If the water is being wasted, the water master is authorized to close the gate to such extent, permitting such water to flow down to others.

The point of diversion, or even the location of the ditch, is almost immaterial in the definition of a water right, as its location must change to conform to the new conditions of channel caused by floods, etc.

In addition to these five fundamental points, the water right certificate will give the name and address of the owner of the right and the name of the stream from which water is diverted.

Much confusion will be avoided if these fundamental principles, essential to the definition of a water right in some of the Western States, are kept clearly in mind.

THE GRANTING OF NEW RIGHTS.

Rights to the use of unappropriated waters are initiated by filing an application with the State Engineer, on forms prescribed by him. Priority of appropriation and beneficial use is the basis for all rights hereafter acquired. These rights will date from the receipt of such application in the State Engineer's office. It is not necessary that maps accompany such application, or that complete information be presented. If the project is extensive, the application will be held for a sufficient time within which to make the surveys, prior to its return for correction. Thirty days is allowed by law for such purposes, and special request must be made, when filing, if additional time is wanted. The application can be returned for correction without loss of priority. When the application is put in proper form and is approved by the State Engineer, it becomes a permit, and the applicant is authorized to proceed with construction.

No right to the use of any public waters can be acquired without making such application and securing a permit. This is also true for the extension or enlargement of existing rights. Riparian owners must comply with the law the same as others. The owner of a spring tributary to a running stream must also make application. A reliable record of all new rights or extensions is absolutely essential for proper State control. It is the basis for new investments, as well as for the

protection of prior rights. To insure the reliability of this record, the following penalties have been provided:

"It shall be a misdemeanor to use, store, or divert any water until after the issuance of a permit to appropriate such water.

"Any person who shall willfully divert or use water to the detriment of others, without compliance with law, shall be deemed guilty of a misdemeanor.

"The possession or use of water, except when a right of use is acquired in accordance with law, shall be *prima facie* evidence of the guilt of the person using it."

No application will be approved for more water than can be applied to beneficial use, and the State Engineer is authorized to limit an application to a less quantity of water than that applied for, if there exists substantial reason therefor. During the past three years all applications for the use of water for irrigation purposes have been limited to 1 cu. ft. per sec. for 80 acres. An application may be denied where the proposed use conflicts with determined rights, or is a menace to the safety or the welfare of the public.

Work must be commenced within one year from the date of approval of the application, and be completed within a reasonable time, as fixed in the permit, not to exceed five years. Additional time after completion is allowed within which the water must be completely applied to beneficial use. Annual proof must be filed with the Board of Control to show progress of work under the permit. If the water is applied to beneficial use within the time allowed, final proof is taken of such fact by the Division Superintendent, and a survey is made in order to ascertain the extent of the use. Based on such proof and information, the Board of Control will grant to the applicant a water right certificate.

This certificate will be of the same form as that issued to early appropriators on final determination of their rights. The form of this certificate has been fully described above.

The fees paid to the State for the privilege of appropriating its public waters are small, but in the aggregate more than equal the appropriation for the office and department of the State Engineer.

Protection of Rights.

The primary object of making water rights definite and certain, through adjudications by the Board of Control, and through permits

issued by the State Engineer, as outlined herein, is to furnish a proper basis for the protection of such rights by the State.

The accomplishment of this object furnishes at the same time the only safe and reliable basis for new investments. Only the more expensive projects remain for development. For this reason no State which fails to provide such a system of water records, and public supervision of stream diversions, can expect to secure the highest development of its water resources. Private capital will not invest without such records and information, and public funds should not be invested, for obvious reasons.

To facilitate the distribution of water, the State has been divided into two parts, over which is placed a superintendent. These divisions are cut up into districts, as rapidly as the various rights are determined and the need thereof arises. On the petition of one or more water users of such district, a water master may be appointed by the Board, on the recommendation of the superintendent.

It is the duty of such water master to regulate the head-gates of ditches so as to insure the delivery of water in accordance with established rights, and to prevent waste. He has authority to make arrests, and to compel the installation of the necessary head-gates and measuring devices. When the water supply falls so low as to be insufficient for all, the ditch of the latest appropriator, having the poorest right, will be closed, so that prior rights may be protected. As the water falls, other head-gates will be closed, and these in turn will be opened in case of a rise in the river.

If water is stored at the head of a stream, and the natural stream channel is used in conveying such water to the place of use, it is the duty of the water master to set all head-gates so as to admit only the quantity to which each ditch is entitled, in order that the reservoir owner may recover at the place of use the water he turns into the stream, less the quantity lost by seepage and evaporation in transit.

Protection of the Public Interest.

Public interest demands that water be put to the highest use. In case of conflict between power and irrigation, the application for the former use can be referred to the Board of Control, on the ground that it is a menace to the safety or the welfare of the public, and the Board has power to direct its refusal, if public interest demands.

Likewise, an application for either irrigation or power can be denied if it is in conflict with the higher use for domestic supplies.

Oregon's Supreme Court has upheld the Board of Control on this interpretation of the law in the case of Cookingham *v.* Lewis.* Recently, the Board directed the refusal of a prior water-power application which menaced the water supply of the City of Nehalem.

Water-power permits have not been issued on streams which are more valuable for irrigation purposes than for power, and where an abundance of power can be generated on the tributaries of such stream above the points where diversions for irrigation are possible. A comprehensive plan for the development of the State's water resources is essential, if waste is to be avoided and the public good promoted. To permit the construction of one or two low dams for the utilization of all the waters of the stream at a point which is the only practical place of diversion for irrigation will greatly retard development. The public, in permitting such power development, is selling its birthright for a mess of pottage. With a 15-ft. dam, 1 000 sec. ft. will generate 1 700 h. p., but, if held available for irrigation, would reclaim more than 300 000 acres of land with storage. The power grant, as far as the public is concerned, therefore, is equivalent to a free grant of 300 000 acres of land. It is true that the power grant may later be condemned, but why grant the franchise and delay the development only to buy the same later at an enormous franchise valuation in addition to its physical value, when the same power could easily be generated at a little greater expense on the tributaries of this stream, where no conflict with irrigation would ever occur. From the standpoint of taxable property, the relative merits of those two conflicting uses would be as thirty thousand is to thirty million.

Typical Water Record, Under New System.—Fig. 1 is a plat showing the clearness with which the character and extent of water rights can be defined for each 40-acre tract when the water is made appurtenant to the place of use. A similar plat for each township in the State is kept in a loose-leaf book, and is indexed for quick reference. On these plats all information relating to water is compiled as it accumulates.

If a prospective settler should write the State Engineer asking if the S. E. ¼ N. W. ¼, Sec. 18, T. 15 S., R. 11 E., had a water right, a

* 114 Pacific, p. 88.

reply could be dictated in a few seconds giving definite information that such tract had a vested right to water for 16 acres from Squaw Creek under Water Right Certificate No. 267. Such definite records make present rights more valuable, encourage settlement, and new development. The issuance of overlapping permits can be avoided. The making of water appurtenant to the land irrigated is the basis for this system.

The plat, Fig. 1, shows three different classes of water rights, as explained below the cut. It is peculiar in that no stream crosses its border, yet the lands will receive water from four different streams.

Lands not shaded must remain forever without water. Lands *B*, *C*, and *D* will not receive water unless the contemplated works are constructed within the time allowed. Purchasers buy with notice of this fact. A published record, by stream systems corresponding to this township plat record, will eventually be issued for gratuitous distribution. It will give the Priority, Purpose, Period, Place, and Quantity for each recorded right, with the name and address of the owner. Such a record of vested rights, together with a published record giving the total available supply for corresponding streams, is essential as a basis for the development of our water resources.

Results Accomplished.

The results accomplished during the first three years of public control in Oregon should demonstrate beyond question the value of the new system of water titles. No serious defects in the law have been discovered. It became effective on February 24th, 1909. Much of the first year was devoted to the preparation of forms and records, and outlining the system. Without a definite and comprehensive system, there could have been no hope of bringing order out of the former chaotic condition of water titles.

Old Rights Determined.

In three years, ninety-one petitions have been filed with the Board of Control, asking for the determination of old rights, so that the benefits of State protection could be had.

Surveys looking to this end have been undertaken on only twenty-three different stream systems. The Board is at present several years behind in its work. The present organization could handle a far greater volume of work if additional funds were available for employ-

ing clerical and technical help. The system is simple, but the filing, indexing, and tabulating of rights, and the correspondence with the numerous claimants, involve an enormous quantity of clerical work which requires office assistants who have had several years' experience in such work.

Thus far, complete adjudications have been made by the Board on fifteen different streams, involving 965 separate rights. These determinations affect title to water for the irrigation or 89 084 acres of land, at a total cost to the claimants of $9 119.

TABLE 1.—WATER RIGHT ADJUDICATIONS—CONFIRMED.

Stream.	County.	No. of rights involved.	No. of rights decreed.	Irrigation Rights.		Fees paid.	Average cost per right.
				Vested rights, in acres.	Incomplete rights, in acres.		
Willow Creek..........	Gilliam and Morrow...	204	163	5 936	355	$2 056	$12.60
Mill Creek................	Union	115	84	2 035	......	659	7.85
Squaw Creek.............	Crook	110	80	7 072	9 408	1 085	13.56
Tumalo Creek............	Crook	149	109	3 058	25 120	854	7.84
Paulina Creek............	Crook	23	17	530	174	193	11.33
Cochman Creek..........	Grant.......	8	5	35	......	25	5.03
East Branch Mud Creek.	Umatilla ...	19	17	532	......	156	9.15
South Branch Mud Creek.	Umatilla ...	19	15	445	......	114	7.60
Goodman Spring Branch.	Umatilla ...	11	6	202	......	59	9.79
Totals	....	658	496	19 845	35 057	$5 201	$10.50

WATER RIGHT ADJUDICATIONS—COMPLETE BUT UNCONFIRMED.

Stream.	County.	No. of rights involved.	Irrigation Rights.		Fees paid.	Average cost per right.
			Vested rights, in acres.	Inchoate rights, in acres.		
North Powder River............	Baker	134	20 496	499	$2 244	$16.74
Butter Creek.................	Umatilla and Morrow	62	7 712	......	1 062	17.13
Cottonwood Creek.............	Malhuer......	4	173	......	58	14.61
Sucker Creek..................	Josephine	87	2 162	2 391	428	4.92
Althouse Creek................	Josephine	20	749	.....	126	6.31
Totals	....	307	31 292	2 890	$3 918	$12.75

Nine of these determinations have been confirmed by the Circuit Courts in the respective counties, and 496 certificates have been issued. The average cost in fees to these certificate holders has been $10.50, and

the total area of irrigated land included in these final records amounts to 54 802 acres.

These nine adjudications have been completed without a single appeal to the Supreme Court. The streams involved are among the most complicated in the State. Heretofore, practically every water case was carried to the highest tribunal, and final decisions were delayed from five to ten years.

Surveys have been completed by the State Engineer and maps have been prepared showing all water right locations on twenty-three different streams, including the fourteen mentioned in Table 1. These surveys include 265 055 acres of irrigated land, or approximately 40% of the total irrigated area of the State, as shown by the recent United States Census. This irrigated area is served by 3 305 miles of main canals, and 2 361 separate diversions. The territory surveyed is scattered over 500 different townships.

The streams, including all their tributaries, which have been surveyed, in addition to those mentioned above, and on which the adjudication proceedings are well under way, are given in Table 2.

TABLE 2.—SUMMARY OF SURVEY MAPS.

Stream.	No. of diversions.	Length of main canals, in miles.	No. of townships covered.	Acres irrigated.
Crooked River	152	259	37	19 854
Rogue River	324	463	46	13 715
Umatilla River	259	397	38	15 902
Willow Creek, Malhuer Co.	175	219	16	9 412
Powder River	791	879	52	85 112
Silvies River	208	228	24	64 130
Lost River	37	25	13	5 420
Bechdoldt Gulch	2	4	2	148
Cherry Creek	3	5	1	225
Totals	1 951	2 479	229	213 918

New Rights Granted.

A total of 2 084 applications for permits to appropriate water have been filed with the State Engineer during the past three years. Of this number, 1 024 have been approved, becoming permits, and more than 400 have lapsed or have been cancelled from the records. These permits involve the construction of 2 500 miles of canal and 163 reservoirs, at a total estimated cost of $32 250 000. These works, if con-

structed, will ultimately irrigate 723 100 acres and develop 111 400 h. p. (theoretical). This area is 37 100 acres in excess of the total area which is now being irrigated, according to the recent United States Census. This remarkable showing is due perhaps largely to the confidence of capital in the new law, and it is the small investors, rather than the large ones, who have produced so startling a result in the aggregate.

The total fees collected to date, by the State Engineer's office, under the provisions of the new law, amount to $41 388.60, which is approximately $11 400 in excess of the appropriation for the department during this period. These fees are in themselves a reasonable guaranty that the proposed works will be constructed in accordance with the permits issued.

Protection of Rights.

There has been only one year's experience in the distribution of water by the water master under decrees of the Board of Control. In every case the people have been well pleased with the results, as compared with the old method of enforcing rights by injunction proceedings before the Courts. The law is a success only in so far as the administrative authorities can distribute the water in accordance with the decree, and at the times when it is needed.

Riparian Rights.

Approximately, 2 000 claims to water have been filed in these adjudication proceedings before the Board. All but four have been based on the theory of appropriation and use. These four claimed as riparian owners, but proved rights by appropriation. To the water user, therefore, the question of riparian rights is a dead issue in Oregon.

The riparian owner, under the decisions of our Supreme Court, cannot secure a decree of any definite quantity of water. He has only a reasonable use in common with all other riparian owners. The water to which he is entitled, therefore, is a fluctuating quantity, impossible to define or to protect. What are riparian lands one day, may be enlarged the next by purchase so as to extend even beyond the watershed of the stream. It is only a matter of time when our Supreme Court will declare that this common law doctrine is not applicable to conditions in Oregon, and therefore never has been the law of this State. Recent decisions tend strongly in this direction. "The rules

respecting the tenure of property must yield to the physical laws of Nature, whenever such laws exert a controlling influence."* The necessities of the people require that water shall be diverted for irrigation and other beneficial uses, owing to the peculiarities of soil and climate.†

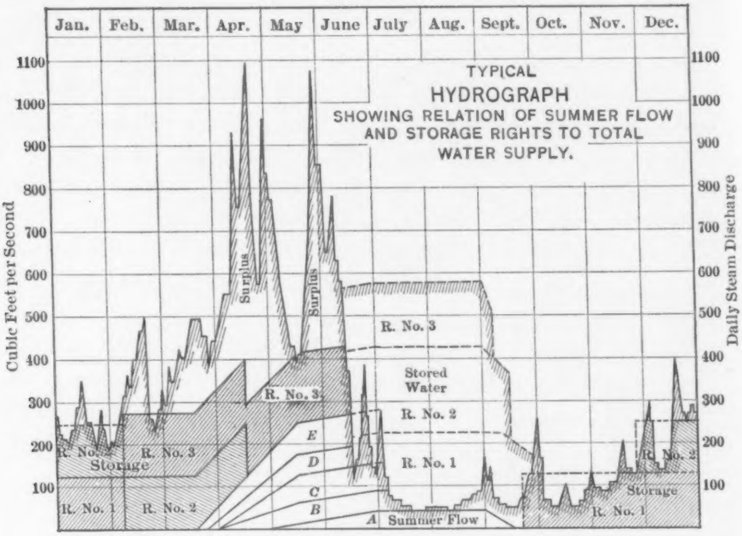


FIG. 2.

The riparian owner must conform to the State laws relative to the appropriation of water. Failure to do so would destroy the system of water titles and make State protection impossible for the water user. Lack of State control will result eventually in unending litigation and community quarrels over water. The new law amounts to an appropriation of all unappropriated waters, for disposal under the new system, and not otherwise. The preservation of the peace and safety of our citizens makes necessary the upholding of this system under the police power of the State.

WATER MAPS.

Water can be measured and shown graphically on a map with as great a degree of accuracy as can land areas. The water map (Fig. 2), or hydrograph, shows at a glance the total volume of water discharged

* Yunker v. Nichols, 1 Colo., 551, 72.

† Walsh v. Wallace, 26 Nev., 229; Twaddle v. Winters, Nev., 85 Pac., 280.

by a stream at a given point; it also shows the discharge for each day in the year, the extent of the regular flow and storage rights which have become vested, and the surplus water available for new appropriations. On it can be shown graphically the character and extent of all diversions, whether from the stream for direct use (Ditches *A*, *B*, *C*, *D*, and *E*), or for storage (Reservoir Feed Canals Nos. 1, 2, and 3). The stored water will be released during the summer, and the extent to which the regular flow of the stream will be supplemented can also be shown graphically. Having careful records of daily stream discharge extending over a series of years, and accurate records of vested rights as determined by the Board of Control under the new system of water titles, it is a simple matter to construct hydrographs which will show with great clearness and certainty the volume of surplus water available for new appropriations. The new law provides a definite method for initiating rights to such surplus water, and grants full protection to such rights during the period of construction.

The Oregon system of water titles is of value because it is complete. Full power for determining old rights, for granting new rights, and for the protection of all rights, is concentrated in one Board. The records are uniform, in simple tables by stream systems, assembled in one central office, where they are thoroughly indexed and digested for quick reference. Every step is taken in harmony with a preconceived plan, and with the purpose of bringing order out of chaos.

The information as to the total water supply is of primary value in encouraging new enterprises. The State is co-operating with the U. S. Geological Survey in the joint expenditure of $56 000 annually on stream gauging work, the making of river profiles in aid of water-power development, and the making of topographic maps. Records of stream flow and of vested rights to water are absolutely necessary for the proper design of new works involving the use of water. We are beginning to realize that it is useless to expend money in the construction of hydraulic works unless water can be delivered with certainty at the time when it is needed. For these reasons, definite State control of all public waters has been undertaken in Oregon.

Interstate Rights.

Just as the lack of a definite system of titles to water within a State depreciates the value of works constructed for the utilization of

water, promotes litigation, strife, and occasionally bloodshed among its citizens, and discourages settlement and new development, so the lack of some interstate or Federal control as to interstate waters affects the territory adjoining such streams in either State, irrespective of whether one or both States have adopted a modern system of control for waters within their borders.

Already interstate troubles are beginning to loom large for the State of Oregon, and it is probable that almost every other State in the Union is similarly affected to a greater or less extent.

A statement of these difficulties will doubtless be of value in showing the need of some legislation in the near future relating to interstate waters.

The United States Reclamation Service has appropriated all the waters of Klamath River and its tributaries in Oregon for the irrigation of several hundred thousand acres of land in Oregon and California. The river, a short distance below, flows into California, where its waters are of value for power development. Power rights in the lower State may in the future be seriously affected by the diversion of water above for irrigation; but a more immediate cause of difficulty is the construction, by the United States, of a dam, at the outlet of Clear Lake in California, which will require three years for the reservoir to fill, when the waters will be turned backward, over a divide, into California, when they have heretofore flowed northward into Oregon and have been used largely for the irrigation of lands in Langells Valley.

Farther to the east, in Lake County, the State of Oregon has under consideration the reclamation of 100 000 acres of land in Warner Valley, under the provisions of the Carey Act. The water for this project will be derived largely from California and stored in a reservoir just over the line in Oregon. The remaining water will come from Nevada drainage. With this project completed, subsequent diversions in California and Nevada may seriously injure prior rights in Oregon.

A 3 000-acre private project on the East Branch of Lost River will depend on water from California, with storage on a tributary, which must be conveyed through California territory to reach the land in Oregon. There are many smaller projects along the southern boundary line which are more or less complicated.

Between Oregon and Idaho several complications now exist. Under

the Carey Act, this State has undertaken the reclamation of 50 000 acres of land in Jordan Valley, the water to be stored on Jordan Creek, just over the line in Idaho. Water can be diverted from this stream just above the reservoir and carried through a short canal in Idaho to Sucker Creek, down which it can be dropped to a reservoir in Oregon. Plans are under way to use this water, supplemented by Sucker Creek, for the irrigation of from 30 000 to 50 000 acres in Idaho.

Diversions from Snake River, in Idaho and Wyoming, in recent years have lowered the water surface so much near Ontario, Ore., that the Wilson Ditch cannot secure its accustomed water supply. Water-power permits granted by Oregon on the Snake River, later, may embarrass Idaho seriously in making diversions for the higher use, irrigation. Diversions from Snake River have affected navigation noticeably on the stream below Lewiston, in Washington.

A large canal has been constructed, tapping the Walla Walla River in Washington; it crosses into Oregon, and irrigates a large area in both States. This stream in Oregon has been in almost constant litigation for thirty years, and a case, started six years ago, and involving more than 400 parties, has just reached our Supreme Court.

Litigation is now in progress on Goose Creek in Idaho, involving Nevada waters passing through Utah. The most complicated case known to the writer is that of Bear Creek, which rises in Utah, flows through Colorado and Idaho, and terminates in Utah, and along which there are extensive diversions in each State and constant trouble over water.

These illustrations should be sufficient to show the real necessity for some legislation. In the Eastern States, no doubt, many similar difficulties have occurred over diversions for municipal water supplies, canal supplies, and for other uses peculiar to that section.

Character of Legislation.

We have paid dearly to learn that county control of waters was impossible. After the sacrifice of much valuable time and money, the destruction of interstate rights, the engendering of strife and bad feeling between States, and the retarding of development in the territory tributary to interstate streams, we may learn that State control of all waters within its borders is just as impossible as county control.

Streams do not follow county lines any more than they follow State lines. Such a division is impossible. We should be able to store water in Wyoming for the irrigation of lands in Malhuer County, Oregon, using the Snake River as a canal through Idaho, just as we can store water at the head of the John Day River in Grant County to be conveyed under State protection through other counties, for use at the mouth of such stream in Oregon. It should be possible to store the flood-waters of the Colorado River in the available basins in Colorado, where no use is possible in such State, and convey this water, under United States or interstate protection, for use in Southern California. The fullest development of streams will lag until such time as some legislation is provided affecting the determination, the granting, and the protection of interstate rights. We cannot expect the United States Supreme Court to work out and adopt some administrative system adequate to the needs in this respect. This is a problem for the legislative branch of the government.

The National Irrigation Congress at Boise, Idaho, in 1906, appointed a commission, of which the writer was a member, to investigate the subject of interstate rights and report the next year at the Sacramento meeting. The views of this commission, as herewith presented, are rapidly being accepted by a growing number of people as the only solution of this vexing problem:

"In view of existing conditions which place the burden of the admininstration of water rights upon the different States, your committee believes and recommends, as do a great many of those answering the interrogatories [replies to circular letters], that the different States affected by interstate streams take some steps looking to a more uniform system of laws affecting water rights, making those laws of each State agree as nearly as possible with those of sister States, and that there should be a general overhauling of all the laws of the arid States as to the matter of water rights, to the end that litigation as to this character of property may be reduced to a minimum, and that the greatest good may come to the greatest number in the matter of the use of water for irrigation.

"While it is true that, in the administration of water rights upon interstate streams by the different States, the right of appeal to the Federal Courts exists, that remedy is expensive, slow, and unsatisfactory. A decision of a Court, once rendered, remains fixed, and only settles the particular questions involved in that case, while the conditions surrounding irrigation on either side of the State line are

constantly changing, and the use of water for irrigation is rapidly growing.

"If there is to be any protection of priorities across State lines, it should be by a Federal administrative system corresponding in character to that needed for the establishing and protection of rights within a State."

"Respectfully submitted,
"ELWOOD MEAD,
"FRANK FREEMAN,
"MORRIS BIEN,
"JOHN H. LEWIS,
"Committee."

After six years of further consideration of this important question, the writer is of the opinion that such Federal administrative system should not be confined to the Western States, but should apply to all territory over which the United States has jurisdiction, and embrace all interstate waters and navigable waters, including the non-navigable tributaries of such navigable streams.

STATE OR FEDERAL CONTROL.

This brings us to a consideration of the question of State or Federal control of waters.

As to navigable waters, there appears to be no question as to the authority of Congress to enact any measure which, in its judgment, is necessary to preserve the navigability of navigable streams, even against any State action.

The Supreme Court of the United States, in the case of United States *v.* Rio Grande Dam and Irrigation Company,* said:

"Although this power of changing the common law rules as to streams within its dominion undoubtedly belongs to each State, yet two limitations must be recognized: First, that in the absence of specific authority from Congress a State can not by its legislation destroy the right of the United States, as the owner of lands bordering on a stream, to the continued flow of its waters; so far at least as may be necessary for the beneficial uses of the Government property. Second, that it is limited by the superior power of the General Government to secure the uninterrupted navigability of all navigable streams within the limits of the United States. In other words, the jurisdiction of the General Government over interstate commerce and its natural highways vests in that Government the right to take all needed measures to preserve the navigability of the navigable water-

*174 U. S., 690-703.

courses of the country even against any State action. It is true there have been frequent decisions recognizing the power of the State, in the absence of Congressional legislation, to assume control of even navigable waters within its limits to the extent of creating dams, booms, bridges, and other matters which operate as obstructions to navigability. The power of the State to thus legislate for the interests of its own citizens is conceded, and until in some way Congress asserts its superior power, and the necessity of preserving the general interests of the people of all the States, it is assumed that State action, although involving temporarily an obstruction to the free navigability of a stream, is not subject to challenge."

If, therefore, under the interstate commerce clause of the constitution, Congress has direct power to exercise control over navigable waters, as far as their navigability is concerned, it must follow by necessary implication that Congress can also exercise such control over the non-navigable tributaries of such stream as is necessary to preserve such navigability.

As pointed out previously, the diversions from Snake River, for irrigation, domestic, and other uses, have diminished the flow in the lower river to such an extent as to affect navigation materially. Likewise, diversions from the tributaries of the Willamette River, which is wholly within the State of Oregon, will affect navigation, by reducing the volume of water, and making dangerous certain rocks on shoals which would not otherwise be dangerous to navigation.

If private capital, or even the Reclamation Service, a branch of the United States Government, should construct reservoirs to store winter waters, and divert the regular summer flow to such an extent as to injure navigation, such diversions could doubtless be enjoined by Congress, or by appropriate action before the Supreme Court. It is high time that an end be put to this system of government by injunction, and that definite legislation be enacted so that capital can invest with safety, under proper Governmental sanction and supervision, in those large undertakings which will bring prosperity and happiness to the present generation.

"As to non-navigable streams within the State, which are not tributary to a navigable stream, it is difficult to find any possible grounds for the theory of State control. Where the United States has disposed of its public land, it has either transferred the water with it to the new owner, or has retained its right to it. There is no opportunity for the intervention of a State claim."*

* Morris Bien, U. S. R. S.

In the original thirteen States, the waters are either the property of the public, or are vested in the riparian owners.

It is true that in 1866 Congress recognized the rights of appropriators, and in 1877 dedicated waters on the public domain to appropriation and use, but no where do we find a definite grant of waters to the State. The nearest approach to a grant to a State occurred in the approval by Congress of the constitution of the State of Wyoming, containing these words, "all waters within the boundaries of the State are hereby declared to be the property of the State." This constitution was "accepted, ratified, and confirmed by Congress." In the absence of the words of an express grant, it is doubtful if the mere approval of a voluminous document will ever be construed by the Supreme Court to be a grant of public waters.

The case of Kansas *v.* Colorado* apparently abolishes riparian rights as to the waters of all States, and at least as to all interstate streams. In this case:

"The citizens of Kansas insisted that large quantities of water were being diverted from the Arkansas River by the inhabitants of Colorado, a large number of whom were claiming as prior appropriators and diverting the water to non-riparian lands, as against Kansas, a riparian proprietor. In Kansas, the modified doctrine of riparian rights prevailed, while in Colorado, prior appropriation was and is recognized as the governing doctrine. The Court refused injunctive relief, and dismissed the bill, stating in substance that if the riparian doctrine should prevail in Kansas as against Colorado and against the non-riparian users whose rights were involved therein, Oklahoma and its citizens lower on the Arkansas River might invoke the same rule in opposition to both citizens of Kansas and Colorado, to their great injury, which doctrine, it is observed, would be ruinous in its effects. The Court, in dismissing the bill, indicated that no injunction would lie until a more substantial injury could be shown, and at the same time found that the interference by a large number of appropriators above in the State of Colorado, materially depleted the flow to the riparian lands of the plaintiff."†

There can be no misunderstanding of the clear intent of the Supreme Court to permit of the diversion of water in an upper State by non-riparian proprietors, even though such diversions materially deplete the flow to riparian lands in the lower State. From the definition of a riparian right, as heretofore given, we observe that it is a property right inseparably annexed to the soil, and passes with it, not

*206 U. S., 46.

† Hough *v.* Porter, 51 Oregon, 409.

as an easement, or appurtenance, but as a part and parcel of the land. It is, therefore, a vested right to private property, which, under the Constitution of the United States, the owner cannot be deprived of without due process of law, nor taken for public use without just compensation. We must assume that this recent, able, and carefully considered opinion of the Supreme Court is not in violation of this section of the Constitution, and that it is applicable alike to all States. Therefore, we cannot escape from the conclusion that the common law doctrine of riparian rights, which has been assumed as the law by the Courts in many of the Western, and doubtless all of the Eastern States, is not the law, and therefore never has been the law, notwithstanding many decisions of the State Courts to the contrary, at least as to interstate waters. If it is not the law as to interstate waters within the State, it is difficult to conceive of any theory on which the State Courts could maintain such doctrine for those streams which are wholly within the State and are not tributary to navigable waters.

In summing up this case, the Court said:

"Regarding the interests of both States and the right of each to receive benefit through irrigation and in any other manner from the waters of this stream, we are not satisfied that Kansas has made out a case entitling it to a decree."

By this language, we believe that the Court had in mind, not alone irrigation, but all possible uses that water could be put to in the upper State, in any part of the territory under the jurisdiction of the United States.

The Courts of many of our Western States, long prior to this decision, had reached this same conclusion, but by a different process of reasoning.

"When the question of water rights was first considered in the State of Nevada, the Court held that the patentee of the government succeeded to all of its rights, and among these was the right to have the water of a stream, theretofore diverted, returned to its natural channel. But this case was over-ruled in Jones *v.* Adams. And in Reno *v.* Stevenson (20 Nev., 269) it was unequivocally declared that the common law doctrine of riparian rights was unsuited to the conditions of that State. In the leading case of Clough *v.* Wing (2 Ariz., 371) it is said that the common law, so far as the same applies to the use of water, has never been, and it is not now, suited to conditions that exist here."*

* Wiel, on Waters, p. 188.

This same line of reasoning can and should be applied to every State of the Union, so that the atmosphere will be cleared for the enactment of a comprehensive system of titles to water for the United States as a whole, for in every State it is necessary to divert water for some useful purpose. If precedent and antiquity determines the law, then this common law doctrine should give way to the civil law of Rome, which is based on prior appropriation and use, and can be clearly traced to the earliest history of ancient times.

If, therefore, riparian rights are not considered as vested rights by the Supreme Court, then the inevitable conclusion must be that water is the property of the public, and that Congress, under the domestic tranquillity and general welfare clauses, has power to adopt a comprehensive system of titles to water which will obviate the present turmoil and confusion as to water rights, and will promote the general welfare of the nation in proportion as the Oregon system outlined herein has promoted the general welfare in Oregon.

Equitable Apportionment of Benefits.

In summing up the conclusions in the case of Kansas *v.* Colorado, the Court, after stating that Kansas had not made out a case entitling it to a decree against Colorado, said:

"At the same time it is obvious that if the depletion of the waters of the river by Colorado continues to increase, there will come a time when Kansas may justly say that there is no longer an equitable division of benefits, and may rightfully call for relief against the action of Colorado."

The question naturally arises as to what constitutes an equitable apportionment of benefits, and who has the necessary authority to decide as to this matter.

There is no question as to the ability of the Supreme Court to decide this question in any case properly before it. To take to the United States Supreme Court every case which is now in controversy, is entirely out of the question. The time and expense involved would compel those suffering injury to endure the wrong rather than appeal to the Court. Such an appeal in most cases is entirely beyond their resources.

If in the Kansas-Colorado case, the Court refused to grant to the plaintiff the required relief, after consuming eight years' time in the

examination of 347 witnesses, the taking of 8 559 typewritten pages of testimony, and the making of 122 exhibits, at a cost of about $200 000, it might be a fair question to ask one who believes in this system of government by injunction, how much longer must Kansas suffer before making a second attempt to secure relief, and what would be the estimate of the cost, in time and money, to complete such second attempt.

This system of government by injunction is disastrous to progress. Let it be assumed that the State of Oregon brings suit against the U. S. Reclamation Service to enjoin the storage of water in its Clear Lake Reservoir, recently constructed in California, and wins; it is easy to picture its effect on future development along our State border. No one would dare to invest without prior sanction from some Governmental authority.

In the absence of any specific guidance in such decree, it can only be assumed that Congress must have power to prescribe by law what shall constitute an equitable division of benefits as to interstate waters. Any other view would make the decree ridiculous, as the Supreme Court could not expect that every interstate water problem would be submitted to it for decision. This is an administrative matter; the Court has decided the law, and it remains only for Congress to provide suitable machinery for carrying the law into effect.

If Congress should decide that the enforcement of the doctrine of priority of appropriation and beneficial use, subject in each case to the consideration by the administrative authorities as to the public interest and welfare, should constitute an equitable apportionment of benefits, it is conceivable that the Supreme Court would uphold such act, on the ground that the question involved is of a political nature and therefore not subject to review by the Court.

State control was not undertaken in our Western States until the patience of the water users had been exhausted so thoroughly by the exasperating delays and expenses of litigation, and the inability of the Courts to distribute the water through injunction proceedings, that they took the law into their own hands, with the result that many murders were committed. It is conceivable that war between States over the proper division of interstate streams may result, before Congress can find time to exhaust thoroughly all the possibilities for debate in this most complicated and involved subject. Decisions can

be found sustaining almost any view. It is easy, therefore, to get hopelessly involved in the details of this subject unless one keeps clearly in mind the ultimate objects to be attained, and proceeds on the theory of the greatest good to the greatest number.

The writer has thus analyzed the question of State and Federal control with respect to navigable waters, including their non-navigable tributaries, non-navigable streams wholly within the State and not tributary to navigable waters, and lastly, the question of interstate streams. He is led to the conclusion that Congress, under the present Constitution, can provide by law a definite system of water titles for the nation, similar to that adopted in Oregon.

State's Rights.

The writer is aware of the criticism that will be urged against this doctrine by the advocates of State's rights, but he is unable to reach any other conclusion which will harmonize with the leading cases and the statutes as we now understand them. It may be of value, therefore, to inquire as to what extent, if at all, an individual State will suffer through the adoption of a national system of water titles.

Prior to the Kansas-Colorado decree, the State of Wyoming, which claims to exercise absolute control over property in water within its borders, would have had good ground to oppose such a system. The State is located on the Rocky Mountain Divide, and all waters originating within its borders flow into adjoining States. Without such national administrative system to restrain over-appropriations in Wyoming, to the detriment of prior appropriators in adjoining States, Wyoming may be considered to have an advantage, being the upper State, and therefore needing no protection from appropriations in adjoining States, while, on the other hand, with a national administrative board to pass upon interstate permits prior to construction, investments could be made without fear of being enjoined by lower States subsequently, and thus, through rapid and safe development, Wyoming would doubtless receive greater benefits from this new system than under the present lack of system. Just as the upper appropriator on a stream needs no protection from lower appropriators, so Wyoming needs no protection from other States; but few States are similarly situated.

If, by such Congressional legislation, the various States which have not yet assumed control of their water resources could be induced by outside pressure to adopt uniform State laws, such States, as well as the country as a whole, would be greatly benefited.

Congress having the power to legislate as to waters, it is difficult to understand how such a body, made up of representatives from the different States, could enact any law which would do great injury to any State or any section of the United States. The control and distribution of waters within the State will be left to the State's administrative authorities. The control of navigable and interstate waters, say in the Columbia River Basin, will be left to the State administrative authorities of this region in proportion as each State is interested, subject, of course, to the general laws, and the rules prescribed thereunder. The Federal administrative board could be composed of all State water authorities meeting annually in convention, or by a smaller board composed of representatives from different sections of the United States.

With the establishment of a national administrative system corresponding to that which has been proven a success in Oregon, Wyoming, and a few other Western States, a comprehensive plan for the development of our rivers in aid of navigation could be outlined and carried to completion without fear of entanglement over interstate rights. The public welfare as to any particular project on an interstate or navigable stream could be considered by the Federal authorities, and, upon approval, construction could proceed with certainty. Such Federal commission should have authority to determine and record all existing rights to water in those States where no such records are available, and should consider the relative value of unappropriated waters to the different States for domestic, irrigation, power, and navigation purposes, and have power to refuse any permit where the proposed use was a menace to the safety or welfare of the public.

Such a National system of water titles, with adequate administration machinery to make it effective, would be of as great advantage to the Nation as the State system is to the individual State. The respective States would derive the same benefit from such system as the individual water user within the State does from the State system. We have priorities in rights to interstate waters, the same as for State waters. We have strong, aggressive, and litigious States endeavoring

through litigation to get some undue advantages over a weaker and upper State in the matter of interstate streams. We have upper States taking more than their rightful share of the interstate water supply, to the detriment of citizens in a lower State, and these citizens are compelled to suffer because of the lack of interest on the part of their State government, or the National government, or some other political or financial reason beyond their control. In other words, the "water hog" is not confined alone to State streams, but is beginning to make himself felt on the larger interstate streams. Here he is so intrenched behind technicalities of the law, and ignorance, due to lack of information as to the effect of numerous diversions on long rivers, that we fail to recognize him. Just as some people within the State, who have an undue advantage in water appropriations, oppose the enactment of State water laws, so we will have States opposing a National water law. Such law would be of no value if it did not restrain diversions in some States for the advantage and protection of others.

The argument of State's rights will be used effectively for a time against the enactment of a Federal water law. The individual State will, in the opinion of the writer, be benefited by such a law, and the Nation, as a whole, receive very great benefits.

Water-Power Policy.

The attitude of the United States toward the policy of water-power development is of particular importance to the Western States, for it is in the West that the greatest amount of undeveloped water power is located. The United States controls the major portion of this power through National Forest Reservations and the withdrawal of all vacant lands valuable for water-power sites. The State, in the absence of Congressional legislation, exercises control over the water, but it is impossible to use this water for power development without access to the stream. Right of way for such purpose cannot be secured without the consent of the United States, as condemnation cannot be resorted to.

Power sites in the National Forests can only be developed under revocable permits, and this is about as serious an obstacle to development as a complete withdrawal. In the streams of Oregon, the U. S. Geological Survey estimates that 3 317 000 h.p. are undeveloped. It

is of great importance to the development of the West that these restrictions in the path of development be removed as soon as possible. As the land is the property of the United States, there is no question as to the power of Congress to enact necessary laws, and such legislation need not await necessarily the adoption of a comprehensive system of water titles for the Nation.

That all power sites remaining on the public domain should be retained permanently by the Government, and be made to yield a material revenue, was suggested recently by the Commissioner of Corporations, at Washington, D. C. He states, further, that:

"Fuel-power will substantially fix the price of all power, because there is practically no considerable area in the United States where water-power can supply the entire demand for power. Of course, water-power cannot be sold above the price of fuel-power. On the other hand, if the price of water-power be fixed by law below that of fuel-power, not all the community, in most instances, can be served with cheap power, and an unfair discrimination must result."

Such policy would seriously injure the West, where water-power is plentiful and coal is dear. It is just as reasonable to argue that cheap coal development of the East should be made to yield a material revenue to the Federal Treasury, so that the West, where coal is dear, may not suffer a material handicap. Besides, such a policy is entirely antagonistic to the conservative theory of preserving to posterity a fair share of our exhaustible coal, oil, and timber supplies.

The views of Secretary Fisher should prevail, as approved by President Taft, and outlined in his message to Congress of February 2d, 1912. That the revenue derived from the development of water-power should be sufficient only to cover the cost of administration of that particular department, any surplus to be expended by the general government for the improvement of the stream, and the benefit of the local community where the water-power site is located.

A more desirable policy, in the opinion of the writer, would be for the States, in co-operation with the United States, to develop this power and supply it at cost plus interest. There is no organization which could secure money at a lower rate of interest than these agencies, and, besides, the greater the scale of the development, the cheaper the original and maintenance costs.

The United States is now engaged in the development of hydro-

electric power, and is selling it at cost to municipalities in the vicinity of reclamation projects for light, heat, and other purposes. The winter rate, on the Minidoka Project of the U. S. Reclamation Service in Idaho, is so low as to permit the use of electricity for the heating of dwellings in competition with coal. The rate for heat, the writer is informed, is $0.50 per kw. per month, with $0.50 allowed to the private company for distributing the power within the municipality. If the States should join in furnishing funds, and in extending this organization, which has already made a success in this enterprise without the least suspicion of scandal, it would only be a few years until the entire United States would be covered with a network of transmission lines. This would be conservation on a grand scale.

For the encouragement of irrigation by pumping in the West, such a policy is indispensable. The cost of electrical power, under the most favorable circumstances, including interest, depreciation, and maintenance for a pumping project, is very high, and a farmer is justified in undertaking reclamation by this method only under the most favorable conditions. Add to these obstacles the chance of paying an increased rate for his power at the end of the usual short-time contracts with power companies, and we have such unfavorable conditions as will compel much of our territory, which could otherwise be reclaimed, to remain arid indefinitely. Irrigation by gravity flow did not become a success in the West until the land and water were united in one by laws making water appurtenant to the place of use. Many communities under common carrier canals became so discouraged, as they watched the charges for water gradually increased in proportion to their ability to pay, that they abandoned their homes. This led to a reform in the water laws.

In 1870, 49% of the population of the United States were farmers. In 1900, only 29% were farmers. This is a most serious condition for our Government to face, if it is to endure. Some of the luxuries of life have come to be considered necessities. These must be supplied to the farmer at such prices as he can afford to pay. We are endeavoring to check this unhealthy trend by supplying good roads, telephone service, rural free delivery of mail, good schools, etc., for the farmer. The writer can conceive of nothing in addition to these which would make country life more attractive than the supplying of electrical

power to the farmer at cost. It can be put to almost immeasurable uses in the home, and on the farm generally.

Private capital will probably never be able to supply the farmer, owing to the relatively long transmission lines required for the limited market. Under State and Federal control, electric rates in cities could be lowered, even though the added cost of country service was assessed on all.

To prevent a threatened water-power monopoly, and to reduce electric rates, the Province of Ontario, Canada, created a Hydro-Electric Power Commission, in 1906, with full power to construct plants and to buy and sell power. This commission is now operating 281 miles of 110 000-volt main transmission lines and 180 miles of distribution lines, buying current from the Ontario Power Company, a private corporation at Niagara Falls, at $9 per horse-power per annum. The rates charged the municipalities to cover cost, vary from $18.10 to $29.50 per horse-power per annum, according to distance, and, at present, twenty-one municipal corporations are supplied with an aggregate of 30 500 h.p.

The two attempts of public developments of power and sale at cost, as here outlined, have been successful thus far. They should prove of value to the Nation in considering public control on a comprehensive scale.

Conclusions.

The distribution of water and the question of water rights are, at best, complicated matters. The present confusion seems to have arisen from the fact that heretofore we have left to the Courts the solution of all difficulties arising from the use of water, when, in fact, they should have been solved by the legislative and executive branches of our government.

The Courts, having no knowledge of the practical difficulties involved in the dividing of streams for the different users, and approaching the problem only after investments had been made to the injury of other investors, have established and maintained a system of law which is entirely repugnant to the diversion and use of water for any but the most primitive uses. Following sound rules of law which had been developed under entirely different conditions, the Courts have refused to the legislative branch the right to amend the Court-made law, on the ground that such statutes would be an inter-

ference with vested rights and in violation of the Constitution. Realizing the extreme necessity of diverting water from streams in the Western States, the Courts, in a few instances, have declared that what they had heretofore considered the law was in realty not the law at all. Now the Supreme Court of the United States, in a decision which applies alike to every State of the Union, appears to indicate that the Courts of all Eastern, and many of the Western, States have been mistaken as to the law of waters, and the way is now clear for the first time in many years for legislation. This legislation should be positive, and should approach the problem from the standpoint of the investor prior to undertaking development. The public interest should be carefully considered prior to the permitting of any new development.

All rights heretofore initiated in good faith for the application of water to any useful purpose should be determined, recorded, and thereafter receive the protection of the law, instead of each user being left to his own individual efforts.

The common error, made by a number of Western States which have attempted to legislate as to waters, should be avoided; that is, to assume that the determination of the facts (priority, purpose, period, place, and quantity), necessary for the establishment of a water right under the law, is a judicial matter, which can only be passed upon by the Courts. Such error would be fatal. These matters should be attended to by an administrative board which has complete jurisdiction of all matters relating to water. Every step thus taken will be in harmony with some pre-conceived plan, and the final results will be of correspondingly more value. Any error in law can be cured by the party injured through an appeal to the Courts. The lack of such appeals, from the administrative officers in Wyoming and Oregon indicates that but a few appeals will be necessary. If, in such system for the adjudication of old rights, the complicated legal rules of pleading and of evidence had to be followed, it is difficult to conceive the cost in time and money, and it is doubtful if a complicated adjudication could be carried through in an ordinary lifetime.

To illustrate, the leading case of Hough *v.* Porter was before the Courts of Oregon for ten years, before a final decree was rendered. There were only forty-five parties to the suit, and an administrative

board could have secured all necessary information and decided the matter in one-tenth the time.

The State of Utah adopted a system of water titles in 1903, where the State Engineer was to make surveys and gather facts for the guidance of the Court in the adjudication of water rights. After nine years, and the expenditure of about $30 000 in the survey of the Weber River, involving about 400 ditch rights, not one right has yet been determined by the Court. This record is in striking contrast with the work of the administrative board in Oregon as outlined in this paper. In only a few years every early right in Oregon will be determined, and will receive the protection of the State's administrative officers.

The subject here presented relates more to the legal than to the engineering side of the water problem, and yet only the engineer can appreciate with full force the importance, to the country as a whole, of having definite and uniform State laws relating to water, supplemented by a National administrative system in harmony therewith. Therefore, he should take a leading part in the reform of State and National water laws. Streams do not follow State lines, and for this reason, these arbitrary political divisions must give way to the more enlightened policy, which is in harmony with the laws of Nature, of treating each stream system as a unit from its head-waters in the forests to its mouth on the coast.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

THE SEWICKLEY CANTILEVER BRIDGE OVER THE OHIO RIVER.

BY A. W. BUEL, M. AM. SOC. C. E.

TO BE PRESENTED NOVEMBER 20TH, 1912.

INTRODUCTION.

The titles of half a score of papers presented to this Society bear the familiar names of great cantilever bridges. Of such bridges in the United States, five exceed the Sewickley Bridge in weight of steel per linear foot, and four exceed it in length of channel span; but some details and erection methods used in this bridge are thought to have sufficient merit and novelty, at least in their application, to justify the hope that a description largely confined to such features may prove interesting.

The Sewickley Bridge has a channel span of 750 ft. from center to center of towers. This is exceeded in the United States by the Blackwell's Island Bridge, 1 182 ft.; the Wabash Bridge over the Monongahela at Pittsburgh, 812 ft.; the Memphis Bridge, 790 ft., and the Beaver Bridge, 769 ft. The Mingo Bridge and the Thebes Bridge both have shorter channel spans, 700 ft. and 671 ft., respectively.

The weight of steel, including the buckle-plates under the roadway floor, is 3.48 tons per linear foot of bridge. This is about 70% of the weight per foot of the Thebes Bridge; 77% of that of the Wabash Bridge at Pittsburgh, and more than 99% of that of the Memphis

NOTE.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

Bridge. When the weight of the concrete foundation and block pavement of the 28-ft. roadway and of the reinforced concrete sidewalk slabs, 5 ft. 7$\frac{3}{4}$ in. wide on each side, is added to that of the steel, it will be seen that the ratio of dead load to total load is probably greater than in any other cantilever bridge in the United States, with the exception of the Blackwell's Island and Beaver Bridges.

The ratio of dead to total loads made it necessary to take great care in providing for the erection stresses, which determined the sections of an unusually large number of both truss members and details. The connections of the floor system and the sub-panel members, as required for the finished bridge, were entirely inadequate to take the traveler reactions or the 30-ton locomotive crane, which weighed 75 tons. The main panel point pins were practically the only details of sufficient strength to take the traveler reactions, and the expense and difficulty of increasing other members and details to carry them would have been very great.

The illustrations will be largely relied on to show the general dimensions, stresses, sections, and ordinary details, but an attempt will be made to present a few points of interest in shop plans, by which economies in weights and shop costs were effected, and particularly some details and provisions for erection which gave very satisfactory results.

Historical.

In 1894 a movement was started for the construction of a bridge over the Ohio River between the Boroughs of Sewickley and Coraopolis. Notwithstanding the fact that there was then no highway bridge over the river above Wheeling, 100 miles from Pittsburgh, the Board of Viewers reported that the probable traffic did not then justify the expenditure.

The movement was renewed in November, 1906, by a petition from the inhabitants of the sections concerned, resulting in an investigation by a new Board of Viewers. This Board reported to the Court that there should be a bridge at this point, but that the cost would be greater than it was reasonable for Sewickley Borough and Moon Township to bear. The approval of this report by the Court and Grand Jury gave authority to the County Commissioners to construct the bridge, if, in their opinion, it seemed expedient. The Commissioners then ordered the construction of the bridge on April 10th, 1908, and appropriated

THE SEWICKLEY BRIDGE OVER THE OHIO RIVER. DISTANT VIEW.

funds for the work during the following year. The principal considerations which resulted in this action were:

First.—That there was no highway bridge over the Ohio River above Rochester, which is 25 miles below Pittsburgh.

Second.—That Sewickley is approximately midway between Rochester and "The Point" (at the junction of the Allegheny and Monongahela Rivers) where both streams are spanned by bridges.

Third.—That Sewickley and Coraopolis are the largest towns on the Ohio River in Allegheny County.

Fourth.—That the bridge would be a connecting link between road systems on either side of the river, comprising many miles of improved roads. At the present time there are 30 miles of improved country roads on the north side of the river and 40 miles of such roads on the south side, tributary to this bridge.

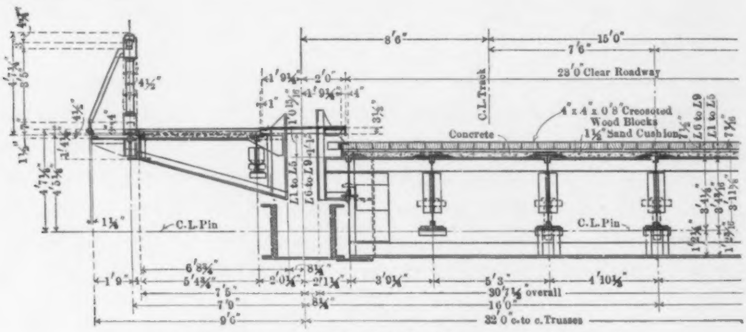
Following the authorization by the County Commissioners, surveys were made and contract plans and specifications prepared under the direction of Mr. J. G. Chalfant, County Engineer. The contract plans comprise 27 sheets, showing general details and arrangements of all parts of the substructure and superstructure. The sheets showing the substructure were, practically, final details. The sheets showing the general details of the superstructure were generally to a scale of ¾ in. to the foot.

SURVEYS.

The preliminary surveys were made during the spring and summer of 1906, and were sufficient to fix accurately the length of the spans and locate the piers and abutments, together with the grades on the approaches and over the bridge. The center line was monumented on both sides of the river during June, 1909, and a triangulation system was laid out. That this part of the work was done with a high degree of skill and accuracy was proven by the satisfactory way in which all parts of the work came together in the field; and it reflects great credit on the County Engineer and his assistants. The triangles closed with errors of one part in 77 000 to one part in 192 000, and to within one or two seconds of arc. The Bridge Company checked the channel span by direct measurement before erecting the south half of the suspended span, and found it to be correct.

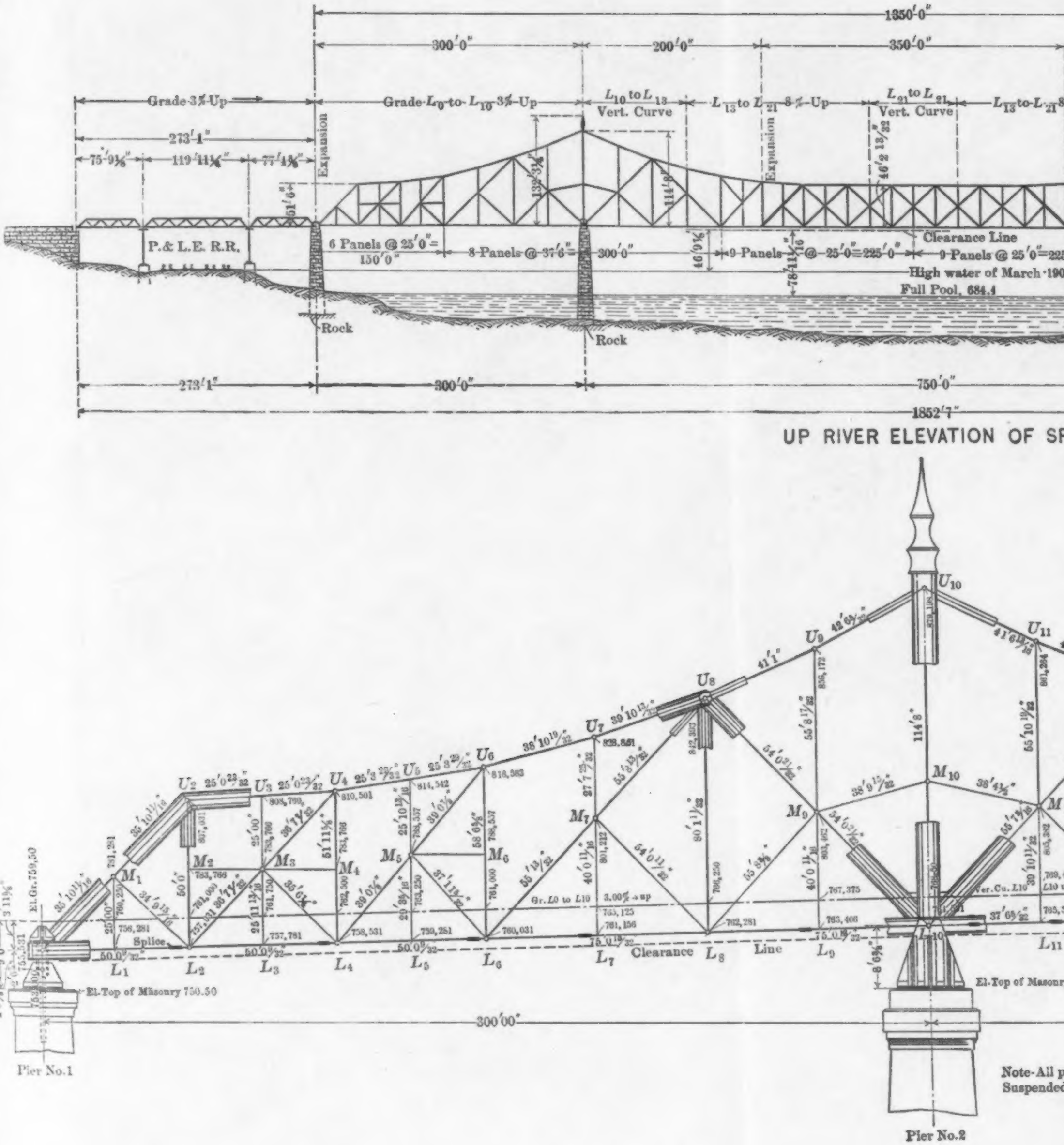
Contract Plans.

Outline.—Plate LXXX is a skeleton elevation of the structure, with dimensions and elevations, prepared in the County Engineer's office. These dimensions were strictly adhered to in the construction. The length of the channel span, the direction of the axis of the bridge in relation to the current at low stage, and the clearance above high water, were practically fixed by the regulations of the Federal Government. The outline and many of the general details of the trusses, shown on the contract plans, were similar to those of the Wabash Bridge, mentioned previously; but, in the latter, the main towers on the cantilever piers are made up with two columns in each truss, connected by horizontal bracing in the plane of the truss, without diagonals; whereas, in the Sewickley Bridge, these towers have only one column in each truss.



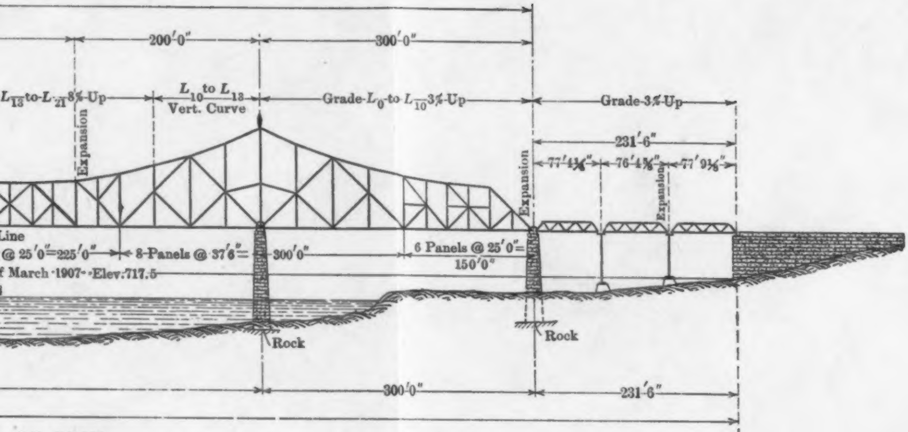
HALF CROSS-SECTION THROUGH FLOOR AT PANEL POINT L_6
THIS SECTION IS TYPICAL, EXCEPT FOR SIDEWALK FASCIAS AND STRINGERS
FIG. 1.

Floor System.—The floor system of the Sewickley Bridge was necessarily very different from that of the Wabash Bridge, as the latter is a double-track railroad structure, whereas the Sewickley Bridge is a highway structure, carrying a 28-ft. roadway with two electric car tracks and two 6-ft. sidewalks on brackets outside of the trusses. The trusses are 32 ft. from center to center. The roadway floor consists of a 4-in. wood block pavement on 1½ in. of sand, with concrete filling, averaging about 3 in. thick, over the ⅜-in. buckle-plates, which were placed with buckles down, and supported by stringers. The sidewalks are reinforced concrete slabs, with an average thickness of about 5 in., supported on two lines of stringers. Fig. 1 is a cross-section of half of the roadway and sidewalk.

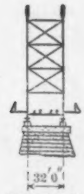


1350'0"
300'0"
200'0"
350'0"
Grade 3% Up
Expansion
273'1"
Vert. Curve
P. & L.E. R.R.
6 Panels @ 25'0" = 150'0"
8 Panels @ 37'6" = 300'0"
9 Panels @ 25'0" = 225'0"
Clearance Line
High water of March 1907
Full Pool, 684.4
Rock
750'0"
1852'7"
UP RIVER ELEVATION OF SP
Pier No. 1
Pier No. 2
El. Top of Masonry 750.50
Splice
Clearance
Line
300'00"
Note-All po
Suspended

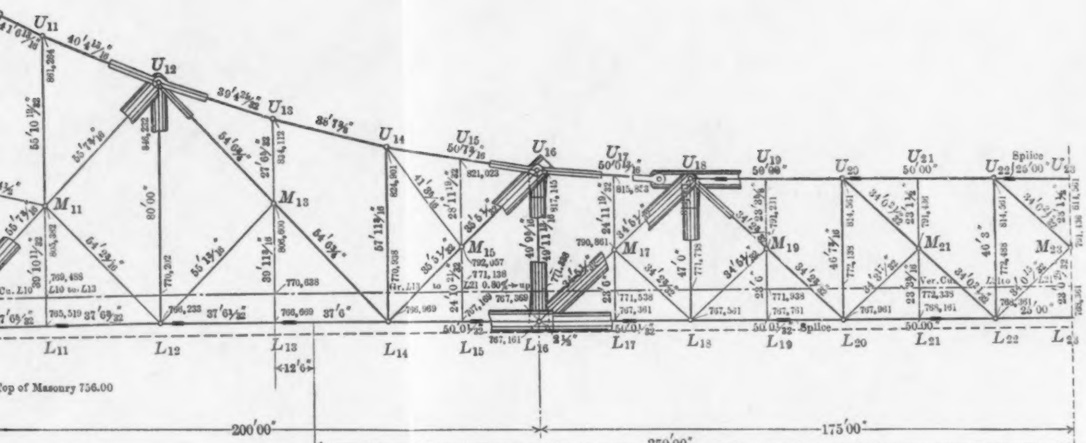
PLATE LXXX.
PAPERS, AM. SOC. C. E.
SEPTEMBER, 1912.
BUEL ON
THE SEWICKLEY CANTILEVER BRIDGE.



OF SPAN



SECTION
AT TOWER



Note-All posts and hangers in Anchor, Cantelever, and Suspended Spans are vertical

Substructure.

The substructure comprises two abutments with wing-walls, four pairs of pedestals, two on each side, supporting the approach spans, the two anchor piers, and two cantilever piers. The foundations of the South Anchor Pier, No. 1, and of the two Cantilever Piers, Nos. 2 and 3, were carried to rock, which was found at elevations of from 30 to 35 ft. below "Full-Pool" water level. The work was done with coffer-dams, without unusual difficulties. The North Anchor Pier, No. 4, was founded on a very dense clay and gravel formation at Elevation 706.4 (about 20 ft. below "Full-Pool" level).

All piers were faced with sandstone and backed with 1:3:5 gravel concrete. The stone for Piers Nos. 1, 2, and 4 came from Wampum, Pa., and for the others from Morgantown, W. Va.

Granite bridge seats, 8 by 11 by 3 ft., were provided for the shoes of the cantilever towers to rest on, each bridge seat consisting of two stones, 4 by 11 by 3 ft., joined on the center line of the truss.

Fig. 2 and Plate LXXXI show the arrangement of the anchorage in Piers 1 and 4. The steel flanges of the double lattice girders were designed to take all the possible stresses, but the steel web members were proportioned for only half of the shear, reliance being placed on the concrete for about 50% of all the shearing stresses.

The appearance of the masonry in the piers is excellent, and, during a period of 9 months, the piers had settled uniformly about $\frac{3}{4}$ in. Mr. Chalfant, his assistants, and the contractor for the substructure, deserve full credit for this part of the work.

Loading.

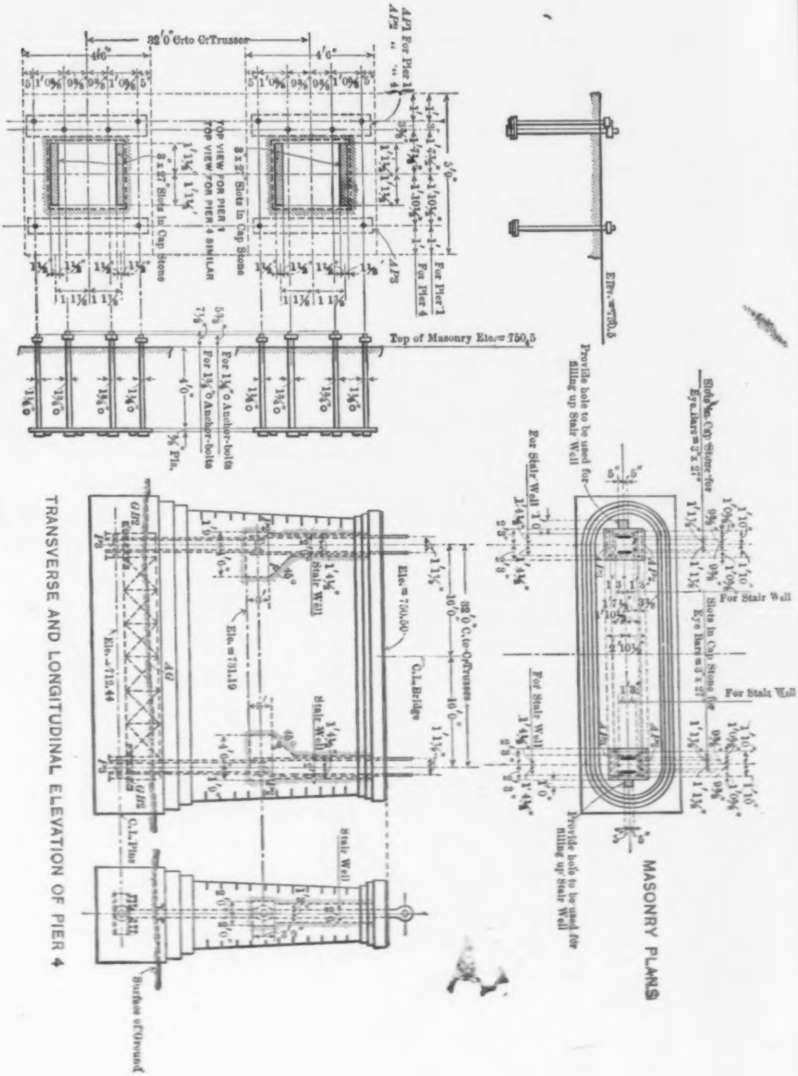
Dead Loads.—The dead loads consist of the weight of the structure itself, including that of the floor system, of any special features peculiar to the structure, and of the street-car rails.

For the contract plans, the dead load was assumed to be concentrated with two-thirds on the loaded chord and one-third on the unloaded chord. In the final computations, made by the Bridge Company, the actual concentrations were computed and used.

Live Loads.—

(A) Floor System.

(a) A uniform load of 100 lb. per sq. ft. of clear roadway and footwalk.



TRANSVERSE AND LONGITUDINAL ELEVATION OF PIER 4

FIG. 2.

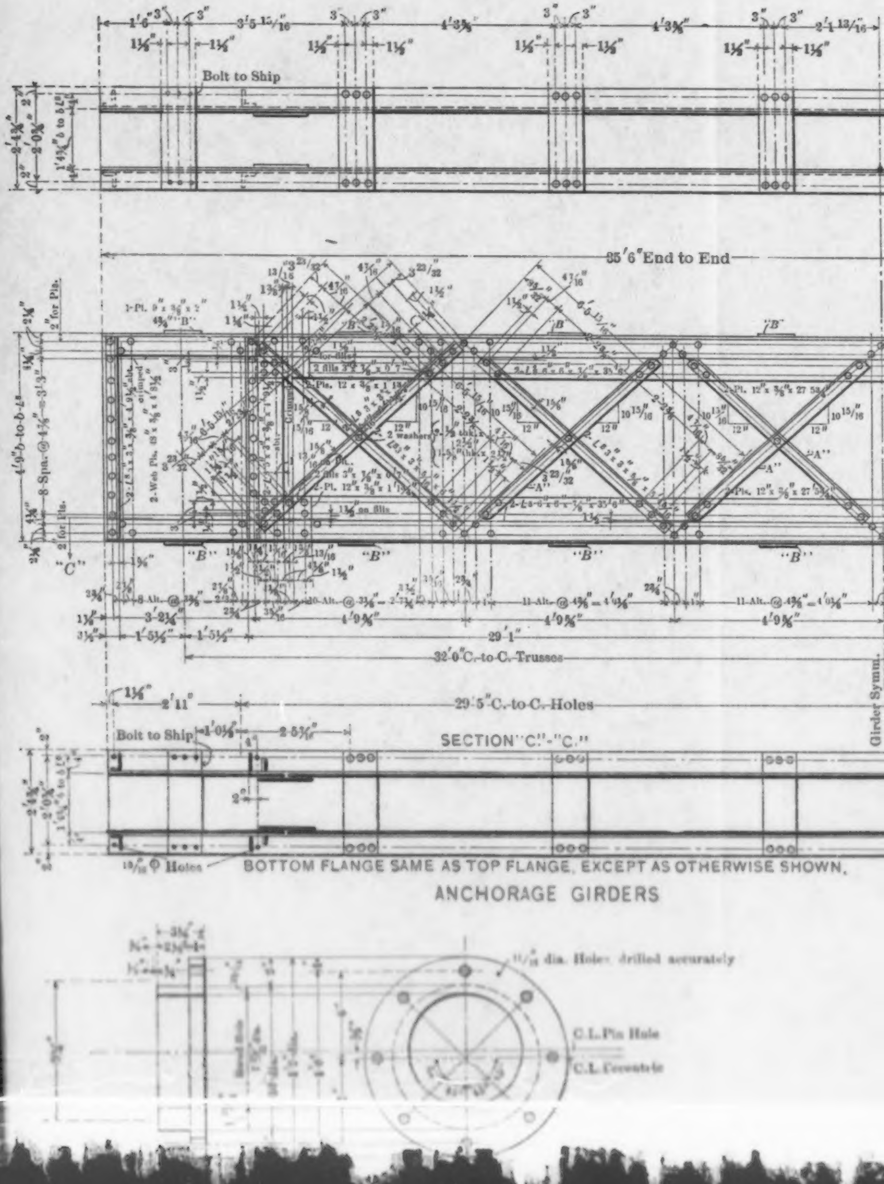
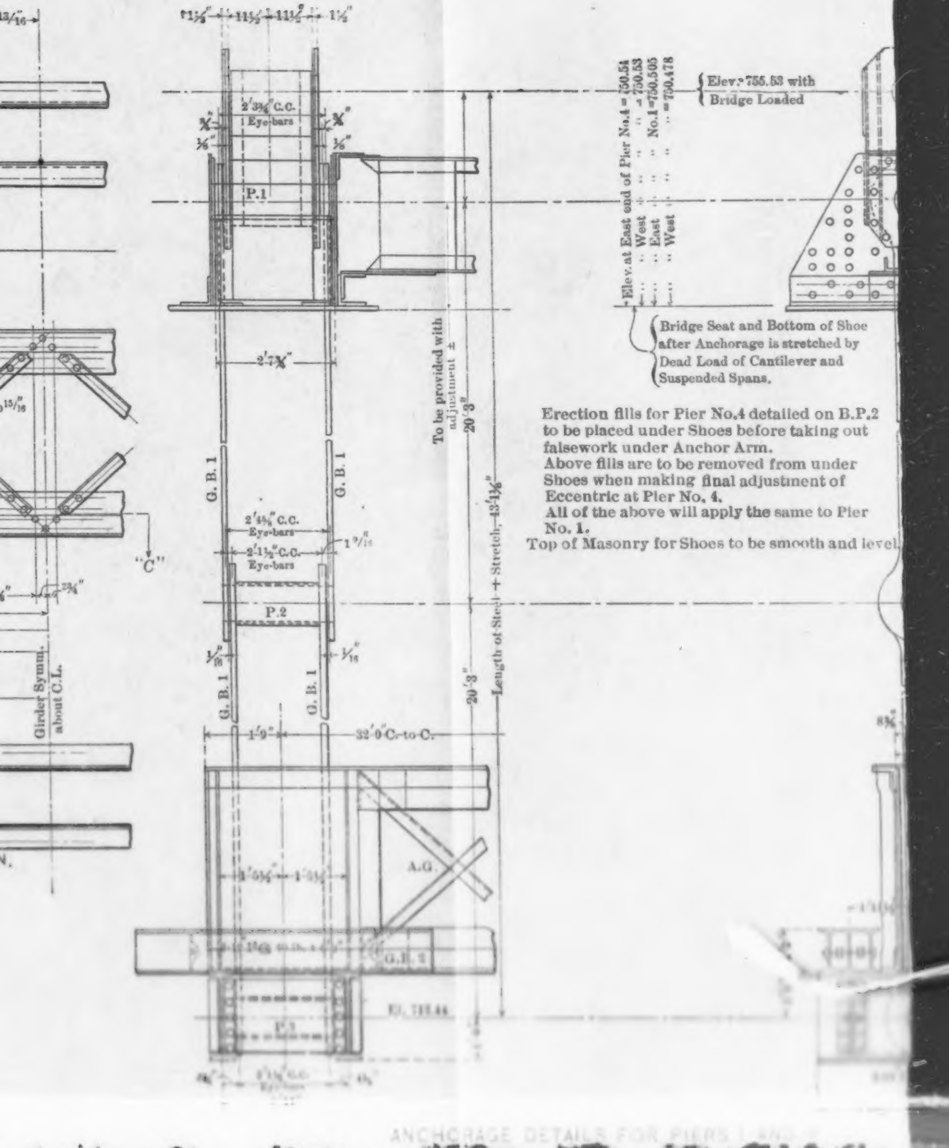
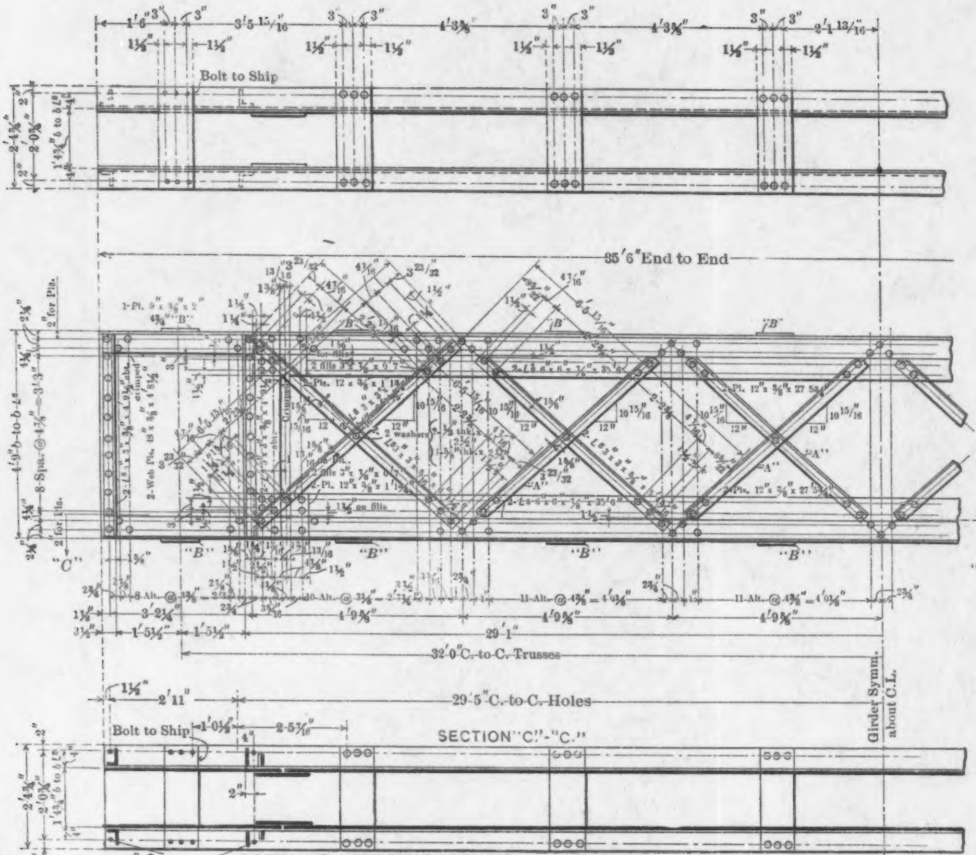

Bolt to Ship
35'6" End to End
32'0" C.-to C. Trusses
29'5" C.-to C. Holes
Girder Symm.
SECTION "C"-"C"
Bolt to Ship
BOTTOM FLANGE SAME AS TOP FLANGE, EXCEPT AS OTHERWISE SHOWN.
ANCHORAGE GIRDERS
C.L. Pin Hole
C.L. Eccentric

PLATE LXX
PAPERS, AM. SO
SEPTEMBER,
BUEL ON
THE SEWICKLEY CANTI

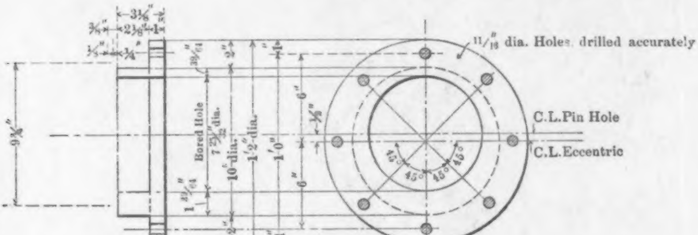


ANCHORAGE DETAILS FOR PIERS 1 AND



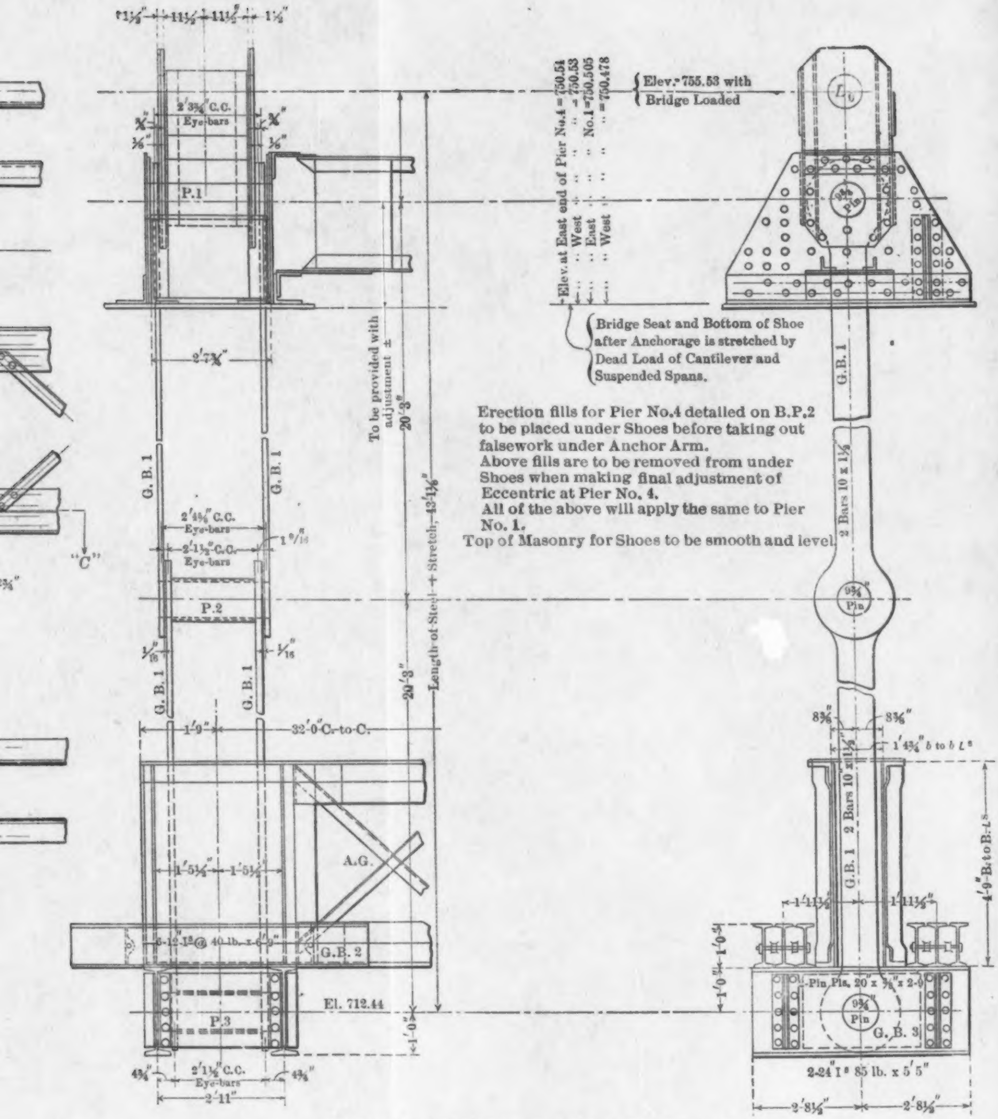
BOTTOM FLANGE SAME AS TOP FLANGE, EXCEPT AS OTHERWISE SHOWN.

ANCHORAGE GIRDERS



CAST-STEEL ECCENTRICS

PLATE LXXXI.
PAPERS, AM. SOC. C. E.
SEPTEMBER, 1912.
BUEL ON
THE SEWICKLEY CANTILEVER BRIDGE.



ANCHORAGE DETAILS FOR PIERS 1 AND 4

(b) A 15-ton road roller arranged as follows: 6 tons on the forward axle and 9 tons on the rear axle; axles 11 ft. apart, rollers 20 in. wide, the two on the forward axle being placed $2\frac{1}{2}$ ft. from center to center, and the two on the rear axle, 6 ft. from center to center.

(c) A wagon load of 10 000 lb. on two axles, 8 ft. apart, wheels 5-ft. gauge.

(d) Two 50-ton street cars, as in Fig. 3.

(B) Trusses.

Suspended, cantilever, and anchor spans, 1 600 lb. per lin. ft. of truss.

Approach spans, 2 000 lb. per lin. ft. of truss.

A wind pressure of 35 lb. per sq. ft. of exposed vertical surface on both trusses, the structure being considered as unloaded.

The maximum stress due to wind added to the maximum stress due to live and dead loads was not permitted to exceed 20 000 lb. per sq. in., reduced by compression formulas for struts.

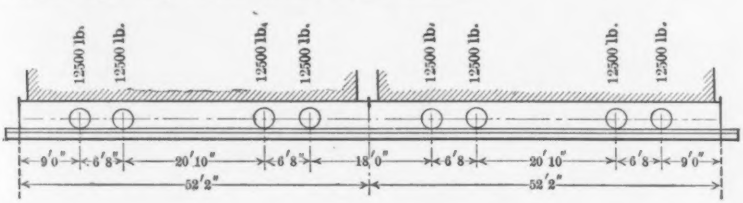


Fig. 3.

Unit Stresses.

(Medium Steel.)

The different parts of the structure were proportioned so that the maximum stress, in pounds per square inch of section, did not, in any case, exceed the following:

	Dead loads.	Live loads.
Tension.—		
Net sections of shapes in bottom chords, main diagonals, and other parts not enumerated, forged eye-bars, main diagonals, and counters	24 000	12 000
Extreme fibers of rolled sections used as stringers and floor-beams, or flanges of plate girders when used as such	12 000	12 000

	Dead load.	Live load.
Net sections of shapes or eye-bars in suspenders, hip verticals, hanger yokes, or other members subject to sudden loading	20 000	8 000

Compression.—

	Dead loads.	Live loads.
Chord members	$24\,000 - 80\dfrac{l}{r}$	$12\,000 - 40\dfrac{l}{r}$
Intermediate and end posts of through and deck trusses, and columns in trestle bents	$20\,000 - 90\dfrac{l}{r}$	$10\,000 - 45\dfrac{l}{r}$
Laterals and rigid bracing	$13\,000 - 60\dfrac{l}{r}$	

l = length of the member, in inches.

r = least radius of gyration of the section, in inches.

No compression members have a length greater than 120 times their least radius of gyration in the direction of possible flexure, except for the struts in the wind-bracing systems, which were limited to 150 times their least radius of gyration in the direction of possible flexure.

Bending.—

On extreme fibers of pins closely packed	25 000

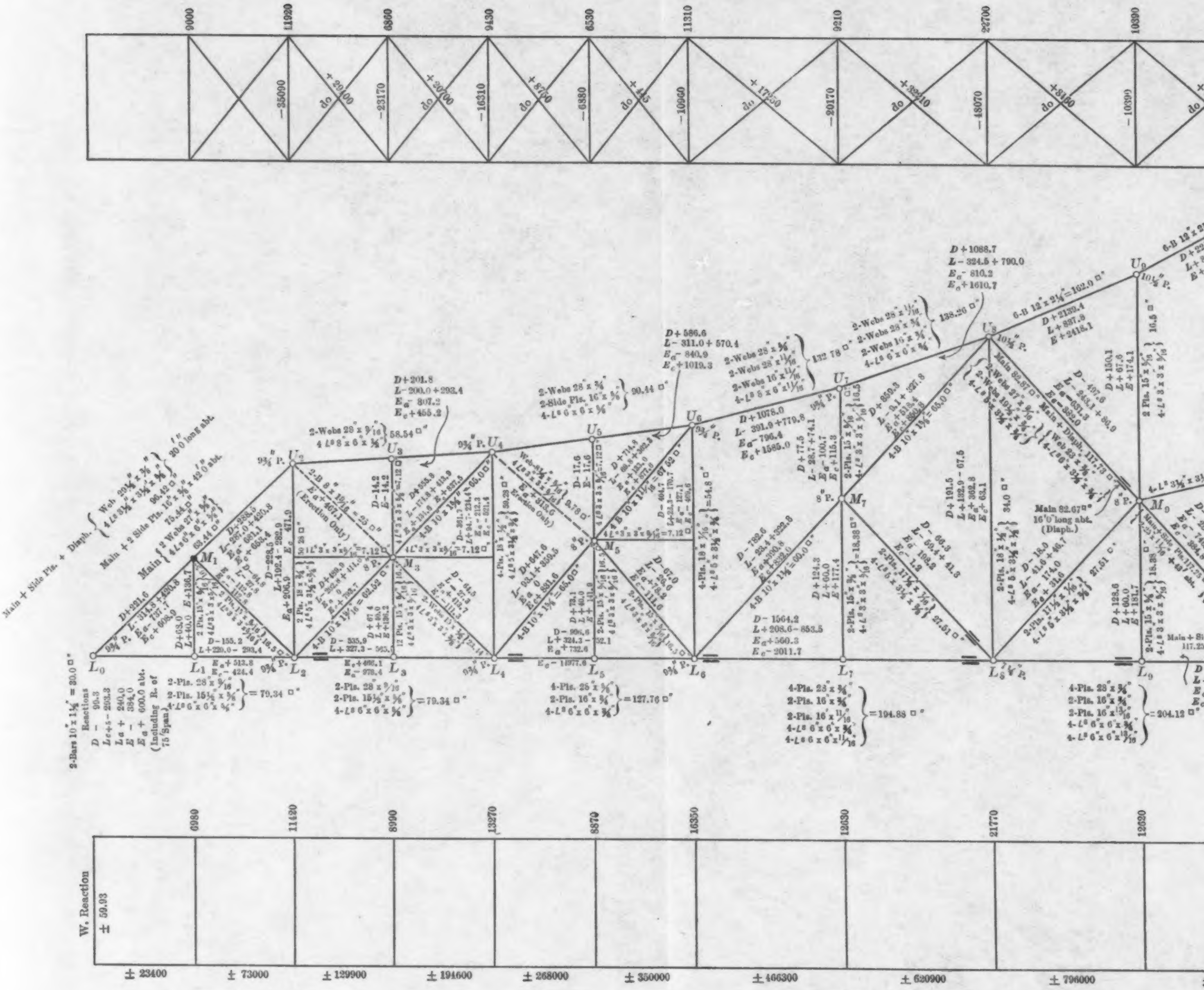
Bearing.—

On pins	22 000
On shop rivets	20 000
On expansion rollers, per linear inch (where D = diameter of roller, in inches)	500D
On bridge-seat masonry—sandstone	500
On bridge-seat masonry—granite	1 000

Shearing.—

On pins and shop rivets	10 000
On webs of plate girders used as stringers and floor-beams	5 000

The unit stresses on the shop rivets of the floor system and their connections did not exceed 80% of the foregoing for all kinds of stress. This applies also to the field rivets in the main girders and trusses. The unit stresses on all the field rivets of the floor system and bolts did not exceed two-thirds of the foregoing for all kinds of stress. The unit

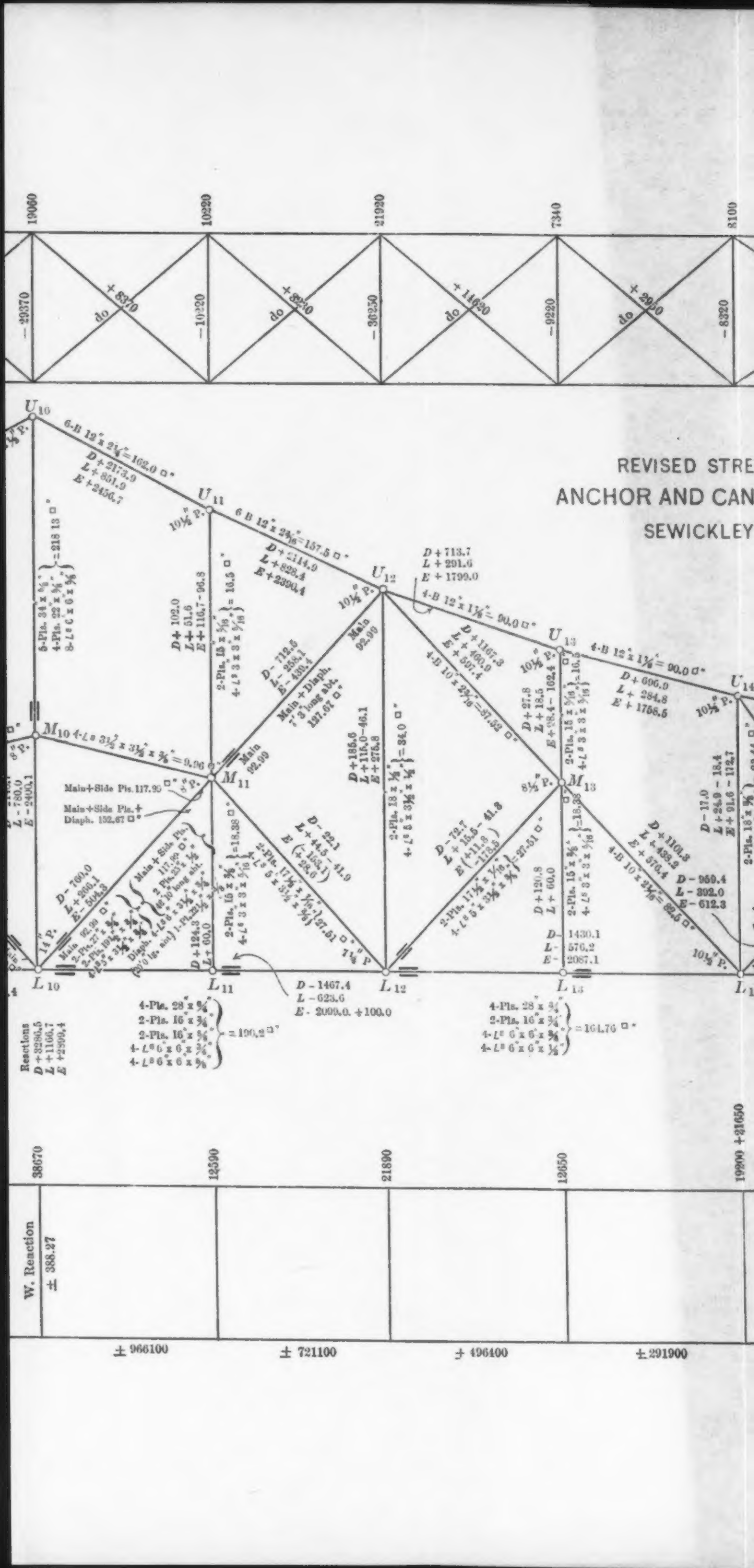


9000
11920
6860
9430
6530
11310
9210
22700
10390
L_0
L_1
L_2
L_3
L_4
L_5
L_6
L_7
L_9
M_1
M_3
M_5
M_7
M_9
U_2
U_3
U_4
U_5
U_6
U_7
U_8
U_9
6980
11420
8990
13270
8870
16350
12030
21770
12030
W. Reaction
± 59.93
± 23400
± 73000
± 129900
± 194600
± 268000
± 350000
± 466300
± 620900
± 796000

REVISED STRESS SHE

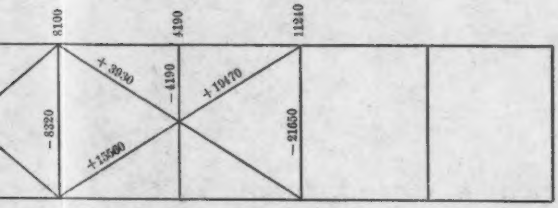
ANCHOR AND CANTILEV

SEWICKLEY BRIDG

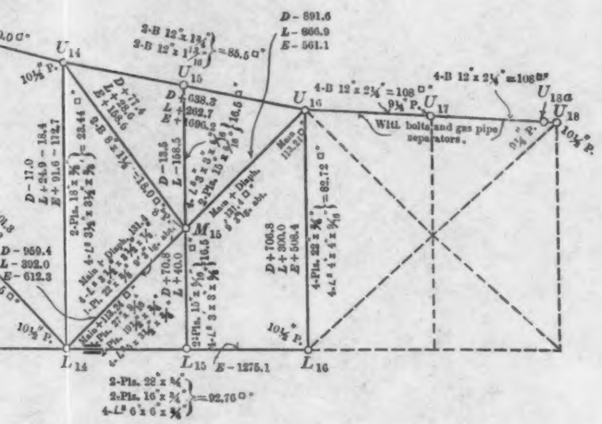


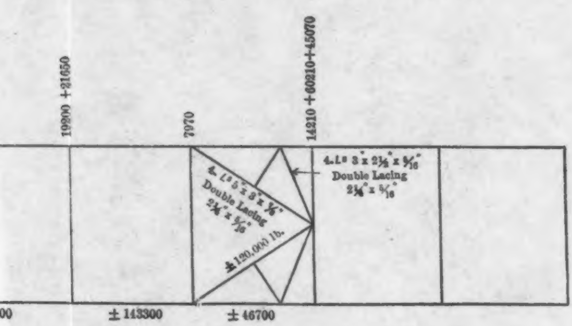
REVISED STRE
ANCHOR AND CAN
SEWICKLEY
19060
10220
21920
7340
8100
− 29370
− 10220
− 36250
− 9220
− 8320
+ 8370
+ 8230
+ 14620
+ 2950
U_{10}
U_{11}
U_{12}
U_{13}
M_{10}
M_{11}
M_{13}
L_{10}
L_{11}
L_{12}
L_{13}
D + 713.7
L + 291.6
E + 1799.0
D − 1467.4
L − 623.6
E − 2009.0 + 100.0
D − 950.4
L − 302.0
E − 612.3
Reactions
D + 3286.5
L + 1166.7
E + 2399.4
38670
12590
21890
12650
19800 + 21650
W. Reaction
± 388.27
± 966100
± 721100
± 496400
± 291900

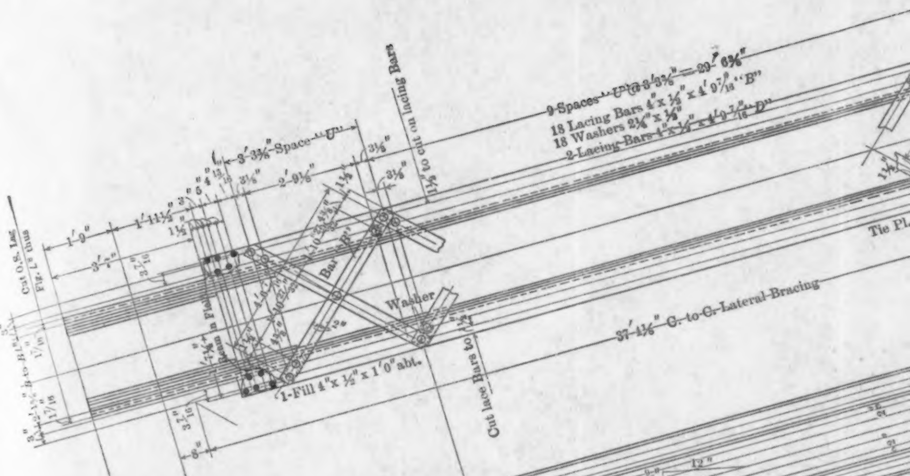
PLATE LXXXII.
PAPERS, AM. SOC. C. E.
SEPTEMBER, 1912.
BUEL ON
THE SEWICKLEY CANTILEVER BRIDGE.



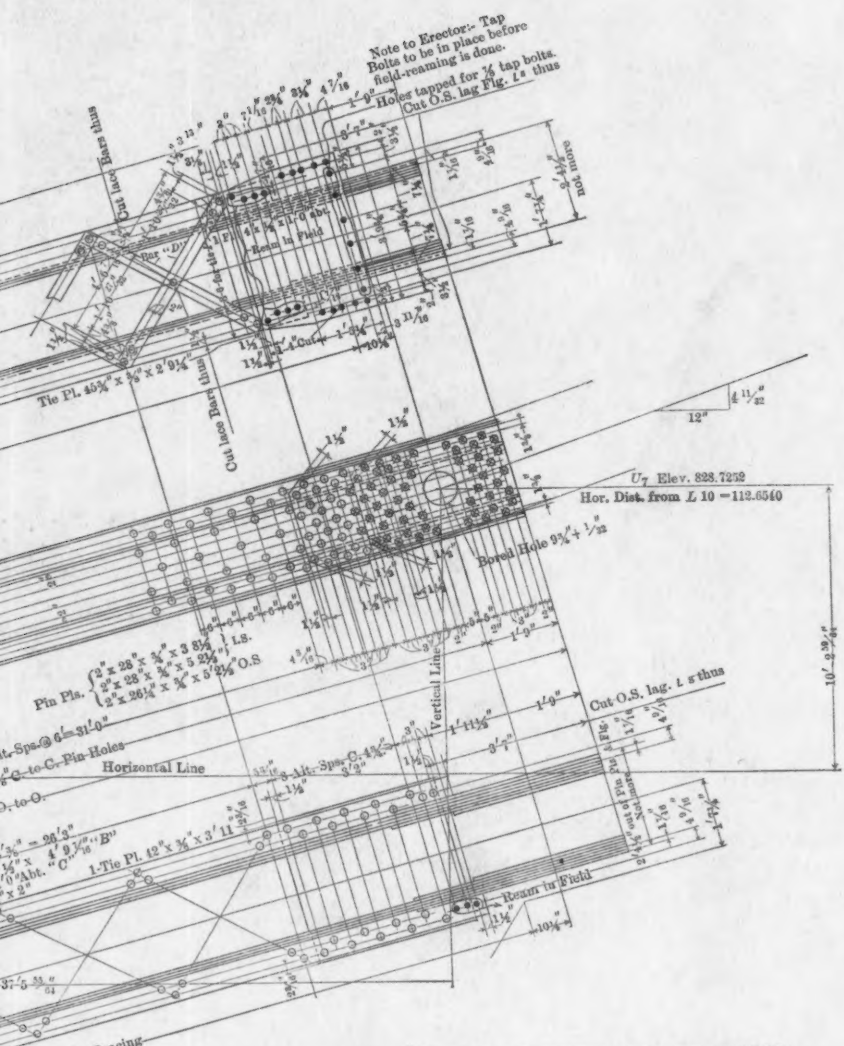
ED STRESS SHEET
ND CANTILEVER ARMS
VICKLEY BRIDGE







9-Spaces "U" @ 3' 3⅜" = 29' 6⅜"
18 Lacing Bars 4" x ½" x 4' 9⁷⁄₁₆" "B"
18 Washers 2¼" x ⅛"
Cut lace Bars to
37' 4⅛" C. to C. Lateral Bracing
1-Fill 4" x ½" x 1' 0" abt.
Washer
Ream in Field
U_6 Elev. 318.4818
Hor. Dist. from L 10
150.1422
4-Flg. Ls 8" x 6" x ¹¹⁄₁₆ x 42' 4⅜"
Webs
2-Side Pls. 16" x ¹¹⁄₁₆ x 42' 4⅜"
Pin Pls.
Bored Hole
42' 4⅜" O. to O.
1-Tie Pl. 42" x ⅜" x 3' 11"
Cut O.S. lag. L s thus
Ream in Field
Cut O.S. Lag. Flg. L s thus



DETAILS OF UPPER CHORD MEMBER
AT $U_6 - U_7$

stresses on the rivets in the lateral and sway bracing were increased by 40% of the foregoing for all kinds of stress.

The other sections of the specifications conformed to the best practice for steel bridgework, requiring sub-punching and reaming, and all details of workmanship were of the highest grade.

The contracts were awarded about July 1st, 1909, to the Fort Pitt Bridge Works, of Pittsburgh, Pa., for the superstructure, and to Adam Laidlaw and Company, for the substructure. The contractor for the superstructure will hereinafter be referred to as the "Bridge Company." Ground was broken on July 21st, and actual work was started on the following day.

Preliminary Work of the Bridge Company.

The Bridge Company concluded to have the engineering work, including the most important of the detailed plans, executed by a specially selected squad of their own engineers and draftsmen, who would devote their entire attention to this particular job, under the supervision of the writer, who was specially employed for this work, and afterward retained as Consulting Engineer during erection. Before this squad could be gotten together, some progress was made by the regular engineering force of the Bridge Company, which took up the work immediately. They first proceeded with a careful revision of the estimated weights and dead loads, and then made graphical check analyses of the stresses. At this point the special squad took charge.

Checking and Revising Computations and Sections.

From the results of the graphical check analyses and other considerations it seemed that the weight of that part of the anchor piers directly over the anchorage might possibly not have a sufficient margin of safety, while, if the greater part of the mass of masonry above the elevation of the anchorage could be made absolutely effective, there would be a considerable excess. As some progress had already been made on one of the anchor piers, any change in the anchorage had to be decided on immediately, or serious delay would result. It was suggested, therefore, that heavy lattice girders, extending nearly the entire length of the pier, should be embedded in the concrete so as to bring practically the entire weight of the superimposed masonry into positive action to resist the uplift. This suggestion was adopted, with the

incidental advantage of reinforcing the piers so that no danger of cracks need be apprehended.

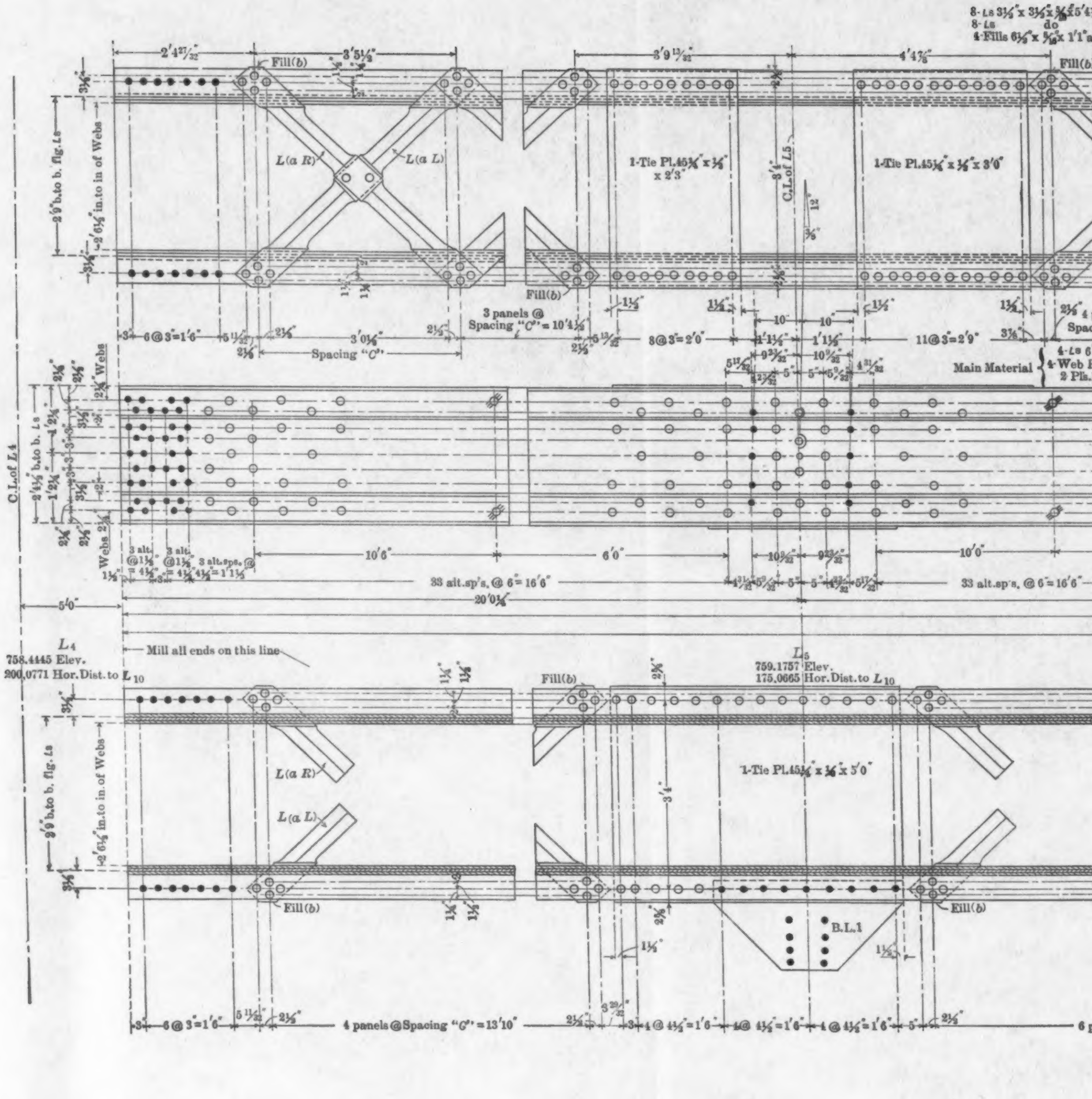
The next step was to check and review thoroughly the dead-load determinations and compute the actual concentrations at each main and sub-panel point (at each point of intersection).

Then tables of stress coefficients, giving the stress in each member for a unit load at each and every point, were prepared and carefully checked. With the tables of stress coefficients and the revised dead-load concentrations, the stresses in each member for the loads at each point were computed and tabulated. These tables were arranged so that the maximum stresses were readily determined by taking summations between limits, which the tables indicated clearly by the plus and minus signs.

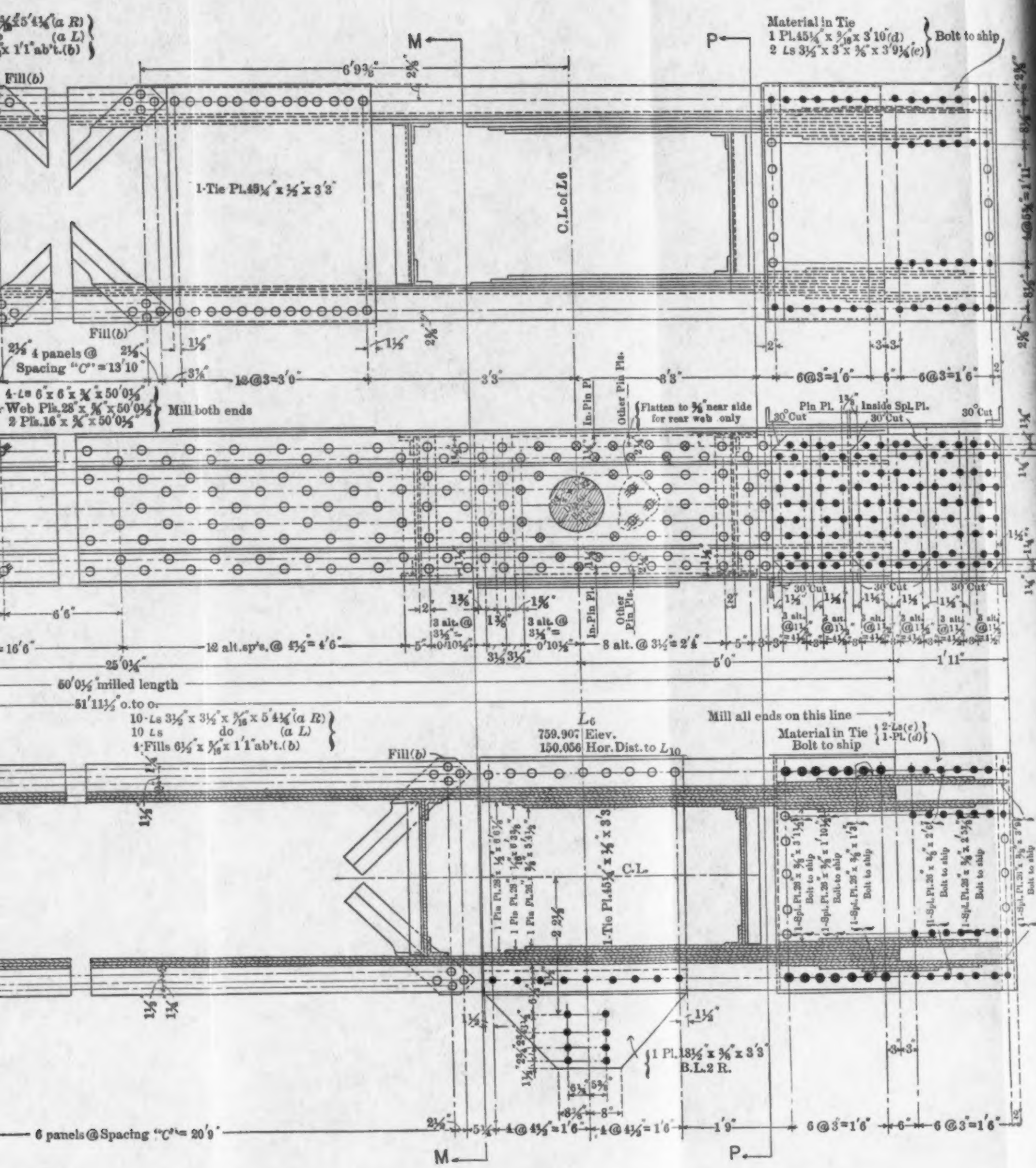
The stresses due to wind were then computed and tabulated, the wind load at each panel being based on an accurate estimate of the exposed areas. The wind shear from the top chord was assumed to go down to the lower chord in the planes, U_{18} to L_{16}, U_{16} to L_{14}, U_{12} and U_8 to L_{10}, and U_2 to L_0, only two panels of wind load being considered as going down the tower, U_{10}-L_{10}.

The final results of this work proved the accuracy of the stress computations on which the contract plans were based, but indicated the advisability of making a few changes, and these were authorized subsequently by the County Engineer.

Top Chord of the Anchor Arm.—The sections and make-up of the end posts and top chords of the anchor arm called for by the contract plans, from L_0 to U_8, consisted of eye-bars (proportioned for the principal tensile stresses), packed inside and outside of a very light riveted member which latter consisted of two 28-in. built channels, latticed top and bottom. These riveted members, while probably sufficient for the slight compression to which they might be subjected in the end post, L_0-U_2, under certain conditions of loading in the finished bridge, were entirely inadequate for the erection loads, should it become desirable or necessary to swing the anchor arm before the cantilever arm was erected. The Bridge Company considered it very important to provide for swinging the anchor arm at this stage of the work. It is also difficult to pack eye-bars in the same member with a riveted chord so that all will work in unison. On presentation of these considerations, the County Engineer approved the change submitted by the Bridge Company,



C.L. of L4
L4
758.4445 Elev.
200.0771 Hor. Dist. to L10
L5
759.1757 Elev.
175.0665 Hor. Dist. to L10
Mill all ends on this line
1-Tie Pl.45¾" x ⅜" x 2'3"
1-Tie Pl.45¾" x ⅜" x 3'0"
1-Tie Pl.45¾" x ⅜" x 5'0"
L(a R)
L(a L)
Fill(b)
B.L.1
Spacing "C"
3 panels @ Spacing "C" = 10'4½"
4 panels @ Spacing "C" = 13'10"
33 alt. sp's. @ 6" = 16'6"
Main Material
2'9" b. to b. flg. Ls
2'6½" in. to in. of Webs



Material in Tie
1 Pl. 45¼" x 5/16" x 3' 10" (d)
2 Ls 3½" x 3" x 5/16" x 3' 9¼" (e)
Bolt to ship
Fill (b)
6' 9⅜"
1-Tie Pl. 45¼" x ½" x 3' 3"
C.L. of L6
4 panels @ Spacing "C" = 13' 10"
12 @ 3 = 3' 0"
3' 3"
8' 3"
6 @ 3 = 1' 6"
4-Ls 6" x 6" x ¾" x 50' 0½"
Web Pls. 28" x ⅝" x 50' 0½"
2 Pls. 16" x ⅝" x 50' 0½"
Mill both ends
In. Pin Pl.
Other Pin Pls.
Flatten to ⅜" near side for rear web only
Pin Pl.
Inside Spl. Pl.
30° Cut
6' 6"
12 alt. sp's. @ 4½" = 4' 6"
8 alt. @ 3½" = 2' 4"
25' 0¼"
5' 0"
1' 11"
50' 0½" milled length
51' 11½" o. to o.
10-Ls 3½" x 3½" x 5/16" x 5' 4¼" (a R)
10 Ls do (a L)
4-Fills 6½" x 5/16" x 1' 1" ab't. (b)
L6
759.907 Elev.
150.056 Hor. Dist. to L10
Mill all ends on this line
Material in Tie Bolt to ship
2-Ls (e)
1-Pl. (d)
1-Tie Pl. 45¼" x ½" x 3' 3"
C.L.
Bolt to ship
1 Pl. 18½" x ⅜" x 3' 3"
B.L. 2 R.
6 panels @ Spacing "C" = 20' 9"
4 @ 4½" = 1' 6"
1' 9"
6 @ 3" = 1' 6"
M
P

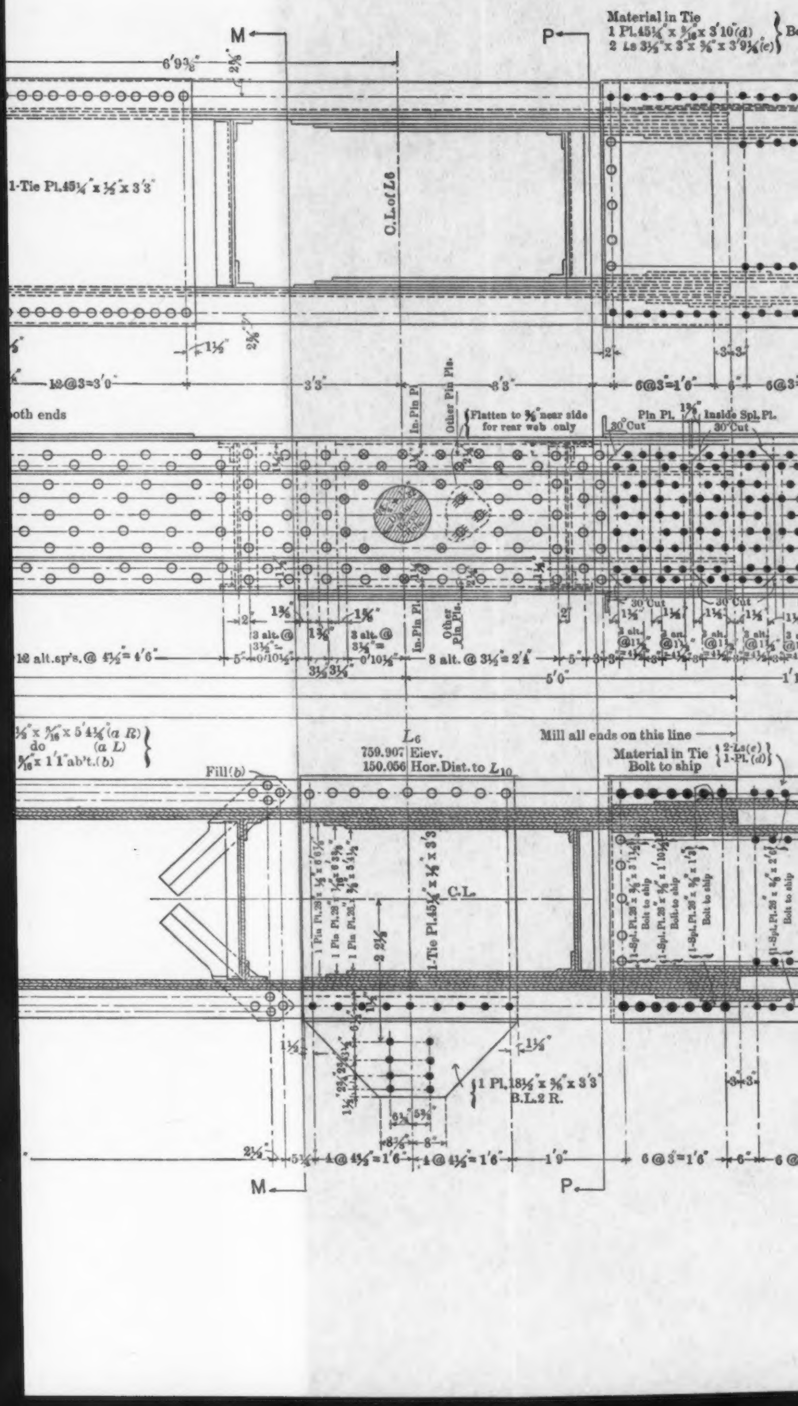
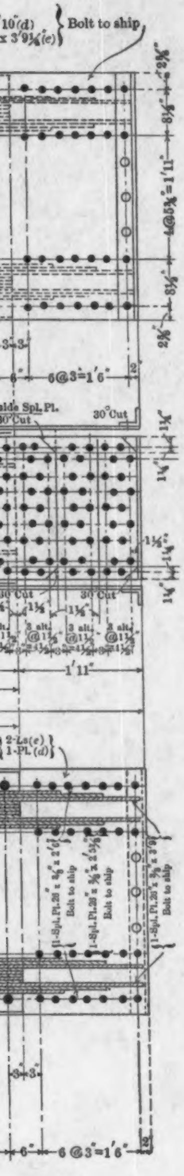
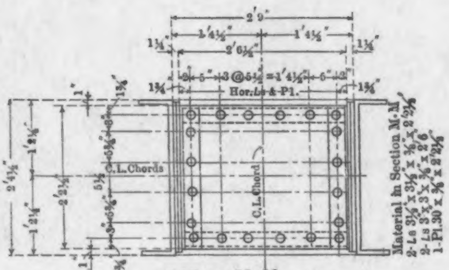

Material in Tie
1 Pl.45¼" x 5/16" x 3'10"(d)
2 Ls 3½" x 3" x 3/8" x 3'9⅛"(e)
6'9⅜"
1-Tie Pl.45¼" x ½" x 3'3"
C.L.of L6
12@3=3'0"
3'3"
6@3=1'6"
Flatten to 3/8" near side for rear web only
In.Pin Pl.
Other Pin Pls.
Pin Pl.
Inside Spl.Pl.
30° Cut
12 alt.sp's.@ 4½"= 4'6"
8 alt. @ 3¼"= 2'4"
5'0"
Mill all ends on this line
L6
759.907 Elev.
150.056 Hor.Dist.to L10
Material in Tie
Bolt to ship
Fill(b)
C.L.
1-Tie Pl.45¼" x ½" x 3'3"
Bolt to ship
1 Pl.18½" x 3/8" x 3'3"
B.L.2 R.
4@4½"=1'6"
4@4½"=1'6"
1'9"
6@3"=1'6"
M
P

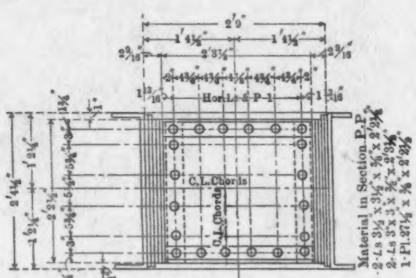
PLATE LXXXIV.
PAPERS, AM. SOC. C. E.
SEPTEMBER, 1912.
BUEL ON
THE SEWICKLEY CANTILEVER BRIDGE.



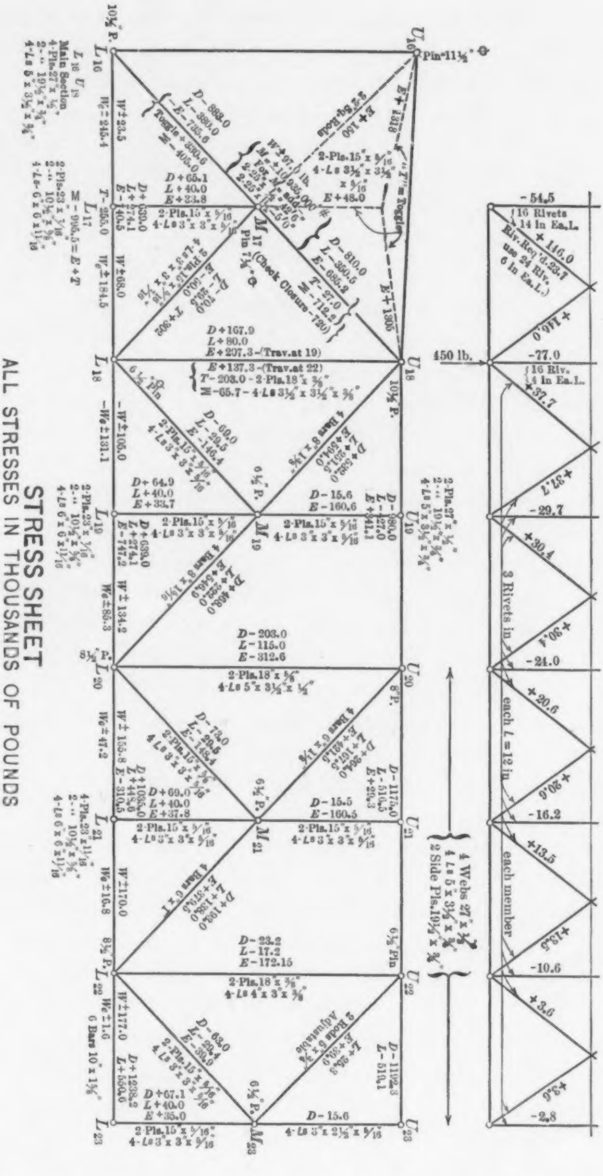
DETAILS OF BOTTOM CHORD
L_4-L_6



SECTION M-M



SECTION P-P



STRESS SHEET

ALL STRESSES IN THOUSANDS OF POUNDS

FIG. 4.

whereby the end posts and top chords of the anchor arms, from L_0 as far as U_8, were made entirely of riveted tension members designed to carry compression during erection and, in the finished bridge, where the stresses reverse toward the end of the anchor arm. Some details of these members will be described later.

Lower Chord Sections.—Some increase in the lower chord sections between L_6 and L_{14} being indicated as advisable, in the light of the revised computations of stresses due to dead, wind, and erection loads, and as an increase in depth of the 28-in. webs would have involved considerable difficulty and delay at that time, these members were changed from two built channels (with four 6 by 6-in. angles) to two built I-sections (with eight 6 by 6-in. angles). This reduced the unsupported length of the top and bottom lattice bars, so that flats could be used instead of channels or angles.

The foregoing changes, together with some in minor members and details, required an entire revision of the dead-load concentrations. This work was expedited and facilitated greatly by the tables of stress coefficients. It was possible for two men to make and check an entire set of revised dead-load stresses for the anchor and cantilever arms in two or three days. As this had to be done several times, these tables saved more than their cost, in the work of computing stresses alone, even if their great value in calculating deflections be neglected. They not only saved time, but made it possible to utilize junior and clerical assistants for a considerable part of the work, which was almost entirely reduced to the simple mechanical operations of slide-rule multiplications and summations by adding machines.

The final stresses and sections are shown on Plate LXXXII, for the anchor and cantilever arms, and by Fig. 4, for the suspended span. The latter are practically identical with those shown on the contract plans, no material changes having been made, except in the diagonals, U_{22}, M_{23}, and L_{22}. The loading of the cantilever arm is shown in Table 1.

Shop Details.

The revised stresses and sections having been approved by the County Engineer, the preparation of shop details and mill order bills was commenced. Each member and each main and sub-panel point was laid out to a scale of 1½ in. to the foot, beginning with the chords of the anchor arm and following with all the other members throughout

the structure, approximately in the order in which they would be required in erection. Considerable pains were taken to treat each detail, however small, on its own merits, applying economic principles of design and shop practice as far as possible, but never losing sight of the risks and costs involved in the problems of erection.

TABLE 1.—LOADING OF CANTILEVER ARM.

Panel points.	DEAD.		Erection (steel only).
	Total.	Top (steel).	
0	55.78	......	40.16
1	93.20	......	61.95
2	107.14	33.49	75.91
3	94.15	14.24	62.91
4	112.27	35.98	81.02
5	103.30	17.60	72.07
6	151.37	45.15	112.32
7	173.12	30.83	126.25
8	193.33	·51.99	146.46
9	197.99	30.62	151.12
10	391.55	193.90	344.68
11	193.28	29.83	146.41
12	184.05	46.75	137.18
13	161.22	17.43	114.35
14	138.03	27.73	98.98
15	108.33	13.51	77.09
16	117.92	22.34	86.68
17 to 23, Inc.	611.19	......	......
17	......	......	72.91
18	......	......	67.65
19	......	......	56.81
20	......	......	65.17
21	......	......	58.37
22	......	......	60.08
23	......	......	54.27

Live load = 1 600 lb. per lin. ft. of truss.
Traveler = 72 500 at U_{11}.
" = 145 000 at U_{12} to U_{22}, at last panel point erected to give maximum stress.
Service track and loaded truck on anchor arm during erection = 100 000 lb. at any one panel point of truss.

Some difficulty was encountered in packing the members on the top chord pins of the anchor arm, due to the rather scant room provided by the contract plans, and also to some increase in thickness caused by the revisions already mentioned. Any increase in the width of the main truss members would have involved changes in practically every member in the bridge, together with a considerable increase in weight, and delay in completing the work. The built members forming the end post and top chord of the anchor arm, principally subject to tensile stresses, were worked out to pack as follows:

The lower end of the end post was designed with forks engaging the pin, L_0, between the webs of the lower chord and the rocker, the in

to in and out to out dimensions of the chord and rocker, respectively, being retained as shown on the contract plans. This fixed the dimension, in to in of webs of the end post, and determined the dimension, in to in of pin-plates, for this member at U_2. This figure was adopted for the dimension, in to in of pin-plates, for both ends of the top chord sections, U_4-U_6 and U_7-U_8. The dimensions, out to out of pin-plates, at both ends of U_2-U_4 and U_6-U_7, was then fixed at ½ in. less than the in to in dimensions of pin-plates on the adjoining members, thus giving ½ in. total clearance for packing. It will be seen this arrangement resulted in the end post and two top chord members being wide and the other two intermediate top chord members being narrow; also, the dimension, back to back of webs, was variable, due to the variation in the thickness of the pin-plates required. In order to minimize the appearance of this difference in the width of the top chord members, those that packed outside were built up with 6 by 6-in. angles, whereas those that packed inside, and were consequently narrow, were built up with 6 by 8-in. angles, the 6-in. leg being always against the web.

The rivet spacing in the webs and pin-plates of these five members was worked out so as to keep the net section through and back of the pin-holes as large as possible and to give similar spacing in all pin-plates. The spacing through the body of the members is 6 in., alternating with a fractional space between the 6-in. spacing and the closer pin-plate spacing. Plate LXXXIII shows the upper chord, U_6-U_7, which is typical.

All the lattice bars, top and bottom, were made in two lengths, without any special bars. This result was obtained by varying the pitch slightly to compensate for the difference in dimensions between gauge lines, and by varying the length of the end tie-plates. This plan was followed throughout the work, an effort being made to keep the lattice bars of the same length for all similar members, and to avoid special bars. Tables showing the items of the lattice bars were then made, with sufficient data for the shop to prepare all the bars, ready for assembling. This was an appreciable advantage, not only in preparing the bars, but in assembling, as the shop plans designated the lattice bars for each member by their item numbers as given in the tables.

Camber and Finished Lengths of Truss Members.—It was desired to camber the bridge so that, under full dead and live loads, it would come to the exact position and elevations shown by the contract dia-

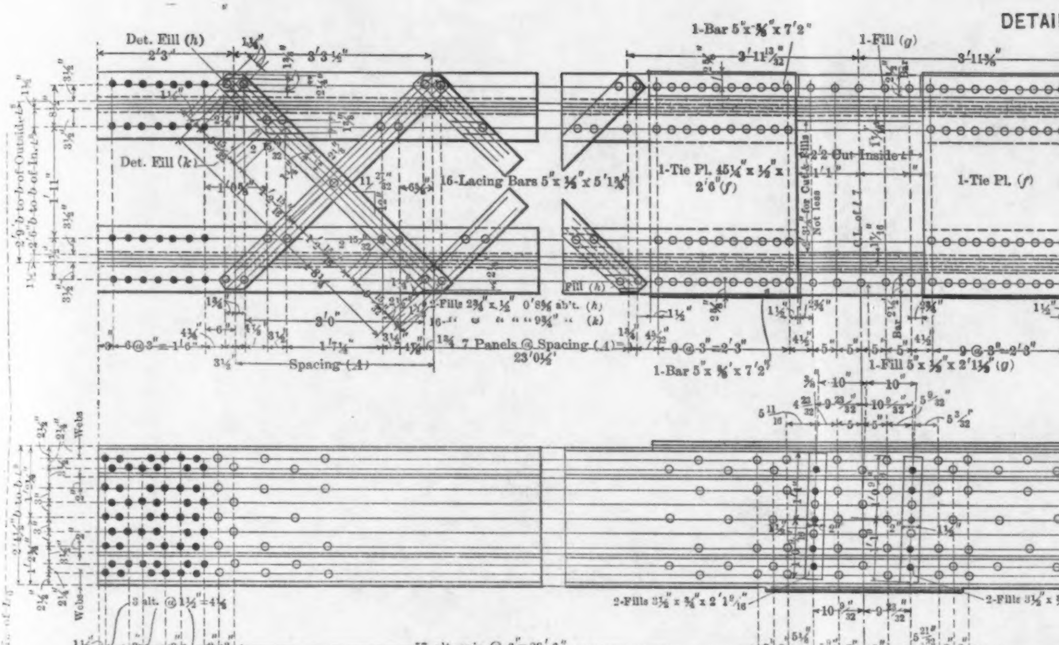
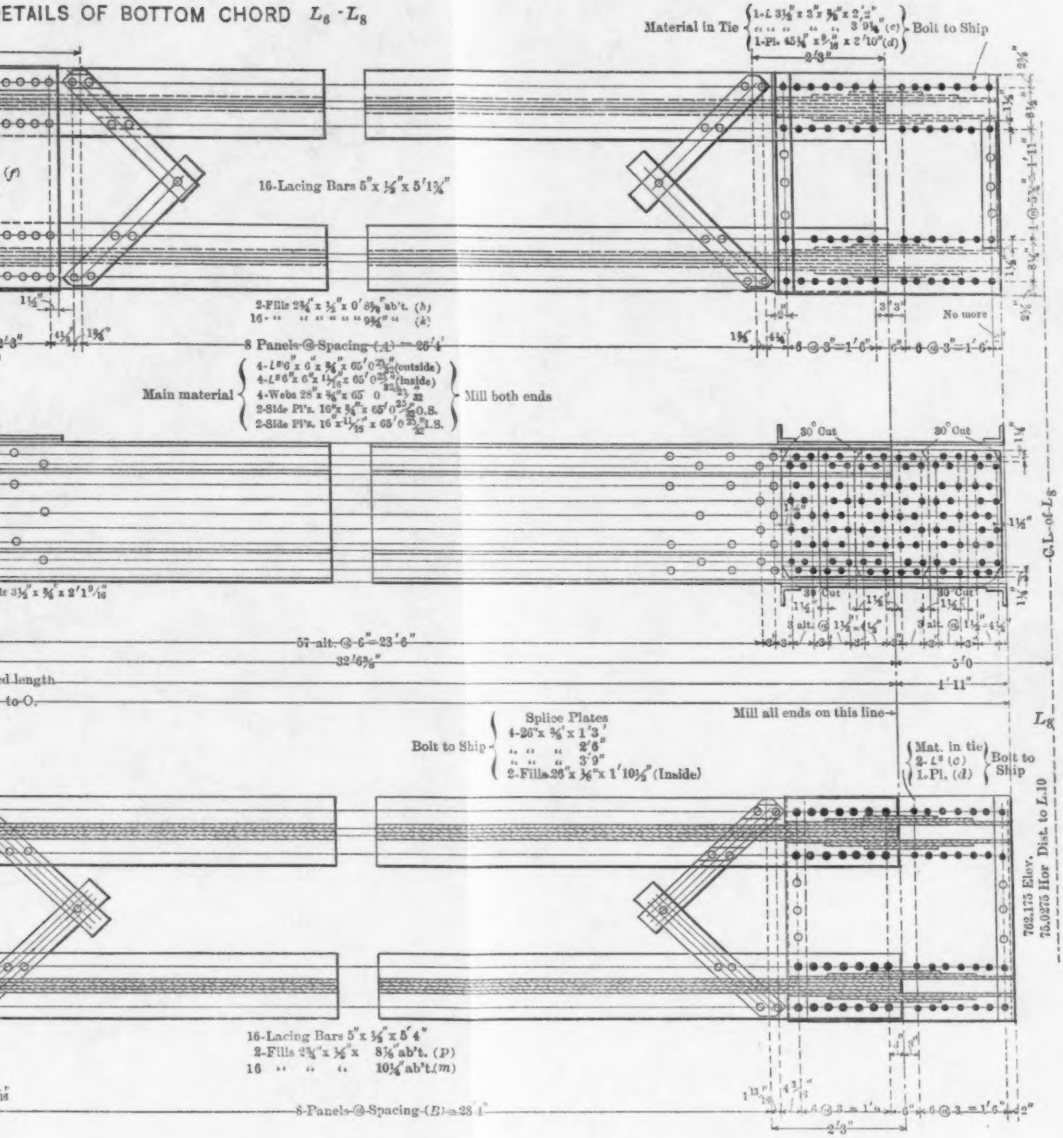

DETAI
Det. Fill (h)
Det. Fill (k)
16-Lacing Bars 5"x ½"x 5'1⅜"
1-Bar 5"x ⅝"x 7'2"
1-Fill (g)
1-Tie Pl. 45¼"x ½"x 2'6"(f)
1-Tie Pl. (f)
7 Panels @ Spacing (A)=
23'0½"
Spacing (A)
1-Fill 5"x ½"x 2'11⅛"(g)
2-Fills 3½"x ⅜"x 2'1 9/16"
57 alt. sp's. @ 6"=28'6"
Mill all ends on this line
761.011 Elev.
112.542 Hor Dist. to L.10
Det. Fill(m)
Det. Fill (p)
1-Tie Pl. 45¼"x ½"x 3'3"
16-Lacing Bars 5"x ½"x 5'4"
7-Panels @ Spacing (B)=24'9⅛"
Spacing (B)
Webs

PLATE LXXXV.
PAPERS, AM. SOC. C. E.
SEPTEMBER, 1912.
BUEL ON
THE SEWICKLEY CANTILEVER BRIDGE

ETAILS OF BOTTOM CHORD L_6 - L_8



gram, Plate LXXX, which gives the "geometric lengths" of all members. The geometric lengths were corrected by an amount equal to the change of length of the member under full load, increasing the geometric length of compression members and shortening that of tension members. The changes of length due to full load were computed for each member with the table of stress coefficients, which are the values of the ratio, u, in the formula, $\frac{p\,u\,l}{E}$. A table was then prepared showing the geometric lengths, the increase or decrease for camber, and the finished length of each member, both in feet and decimals and in feet and inches, and the finished lengths for the stringers, which, being shorter than the finished lengths of the lower chord members, were not riveted to the floor-beams until after the lower chords were under a considerable compressive stress. For erection purposes, and to carry the locomotive crane in setting the falsework bents and floor members, the stringers were supported on shelf-angles with stiffeners proportioned for the erection loads. This table, having been made and checked carefully, assisted materially in avoiding errors in the preparation of the detailed plans.

Provision for Adjustment at the Anchorage.—The lower anchor-bars, up to the first pin below the top of the masonry, were embedded in the concrete, but above this pin an open well was left to permit of some lateral movement and adjustment. After the erection was completed, it was planned to fill the well with concrete through a hole left for the purpose in the top of the pier, completely embedding the anchor-bars. The rocker links connecting the pin, L_0, in the bottom chord with the pin in the upper end of the anchor-bars were not bored until after the anchor-bars were in place. Elevations were then carefully taken on the upper pin-holes in the bars, and the rockers were bored to such lengths as were required to compensate for errors in setting the anchorages. The actual variations were small fractions of an inch.

The $9\frac{3}{4}$-in. pin, engaging the upper end of the anchor-bars and the lower end of the rocker, was made with a reduced diameter of $7\frac{3}{4}$ in. at each end where it passed through the webs of the shoe. The pin-hole in the shoe was bored to a diameter of $10\frac{1}{32}$ in. There was then inserted at each end of the pin, and engaging the webs of the shoe, flanged bushings, bored with $\frac{1}{2}$ in. of eccentricity. As the maximum erection uplift, with the traveler near the center of the suspended span, was

nearly equal to the maximum uplift for the finished bridge, with live load covering the cantilever arms and suspended span only, the eccentrics were adjusted and tap-bolted to the webs of the shoes when the traveler was at its extreme position. This adjustment proved to be efficient as well as extremely simple and inexpensive. It was well adapted for the conditions in this case, but would probably not be available in others.

Lower Chords.—In detailing the lower chords, an effort was made, with considerable success, to use a standard rivet spacing throughout, notwithstanding the variable panel lengths and camber increments. Plates LXXXIV and LXXXV show the bottom chord members, L_4-L_6 and L_6-L_8. As may be seen, the rivet spacing, for a distance of 7 ft. 3 in. on either side of the main panel points and 15 in. and a fraction on either side of the sub-panel points, is fairly close, with thirty-three alternate spaces of 6 in. in the 25-ft. panels. (In the 37 ft. 6-in. panels there are fifty-seven alternate spaces of 6 in.) The twelve alternate spaces of 4½ in. on either side of the main panel points provided for all the lower chord splices in the cantilever and anchor arms except two.

The splices in the lower chord were designed for about 50% of the strength of the members, and consequently increased in length toward the cantilever pier, L_{10}. The rivet spacing in the splices was 1½ in., alternating, which interspaced in the 4½-in. alternate spacing without changing the stagger. This permitted a considerable similarity in the rivet spacing of the different chord members without carrying the 1½-in. spaces beyond the required length of the splice-plates at any point, and made the spacing similar in all splice- and pin-plates. As the bottom chords were parallel with the grade of the roadway, and the intermediate posts and hangers were vertical, the riveted connections at the sub-panel points had a bevel equal to the grade, requiring an odd space on each side of the sub-panel points. This space was also made to serve the purpose of taking up the variation in length of the member due to the camber increment and to the angle between the axis of the member and a horizontal line, thus concentrating all the fractional spacing at one point.

Lower Chord Lattice.—The contract plans required the lower chords to have a double lattice, at top and bottom, made of 6-in. channels, at 10½ lb. per ft., for the anchor and cantilever arms, and of 5-in. channels, at 6½ lb. per ft., for the suspended span. This was changed

SPECIAL BENTS FOR FA

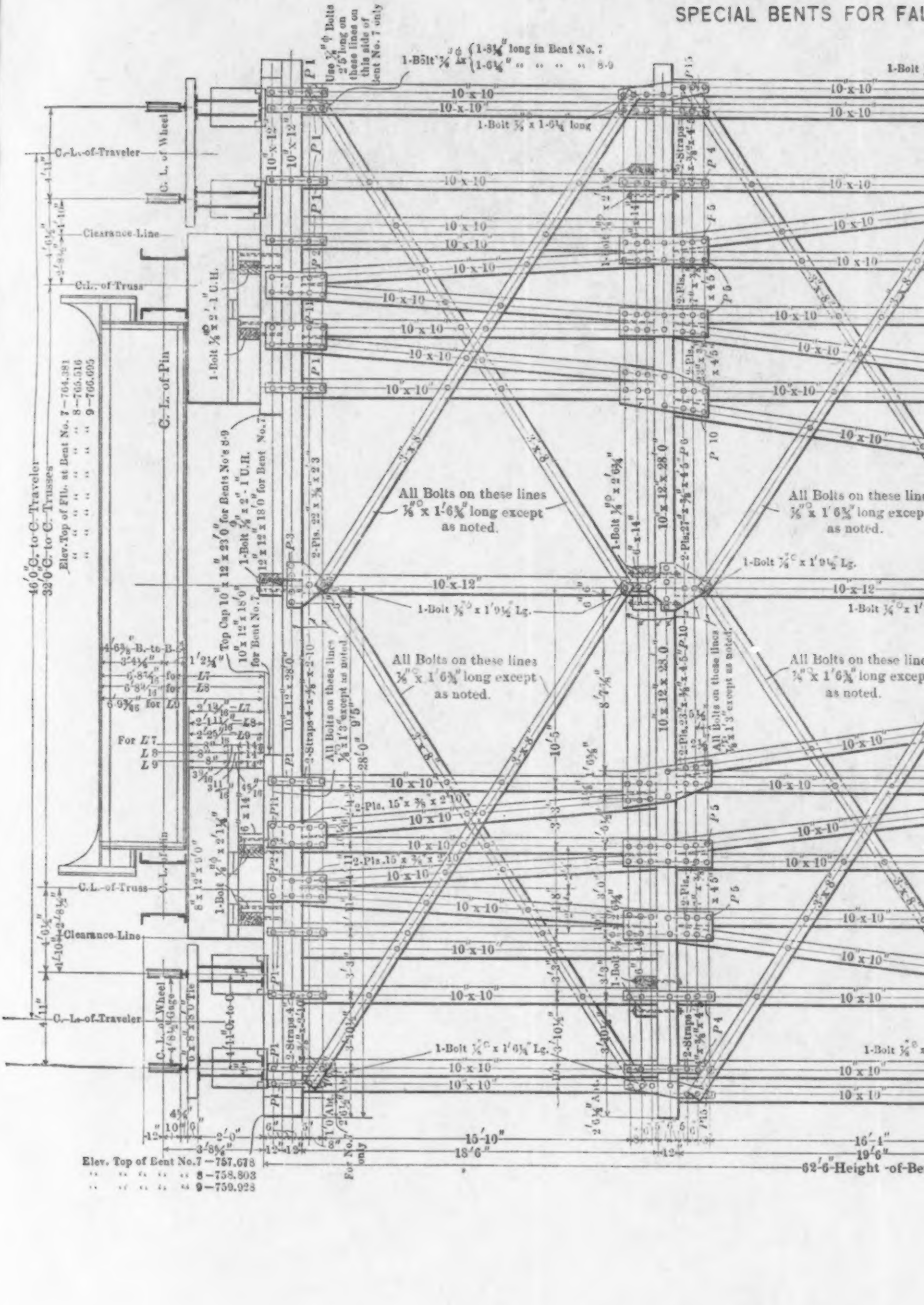
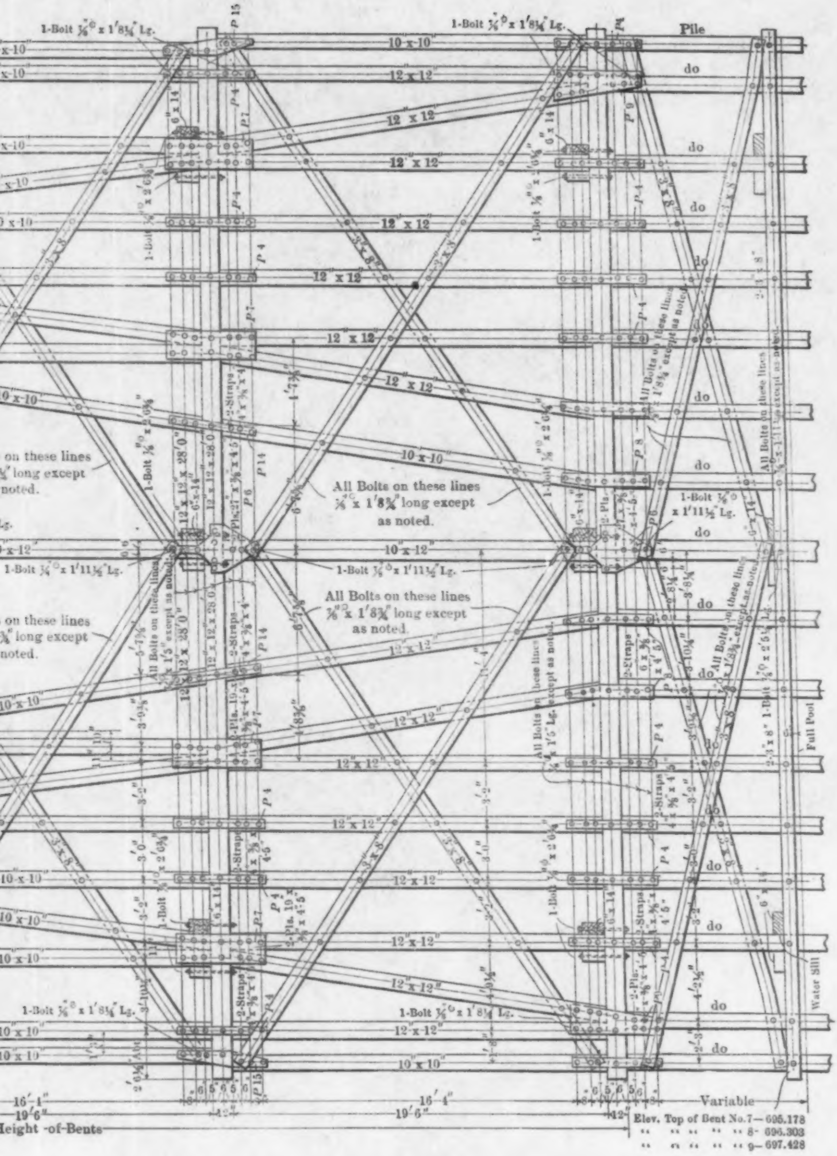


PLATE LXXXVI.
PAPERS, AM. SOC. C. E.
SEPTEMBER, 1912.
BUEL ON
THE SEWICKLEY CANTILEVER BRIDGE.

FOR FALSEWORK.



to 5 by ½-in. flat bars in the lower chord sections between L_6 and L_{14}, due to the change from four to eight angles in the main chord sections, as has been mentioned. In developing the details, it was found that the 5-in. channel was not wide enough to accommodate a sufficient number of rivets, and it was also considered that the metal of the webs was entirely too thin. The effort to find a satisfactory substitute, without increasing the weight, resulted in the use of single angles flattened at the ends where they connected with the main member and at the center where they crossed each other. Experiments showed that, when the ends were flattened out, the center of gravity of the angles was in the plane of the under side of the flattened portion.

Four full-sized comparative compression test pieces were then fabricated and tested. A 3½ by 3½ by $\frac{5}{16}$-in. angle was compared with a 6-in., 10½-lb. channel, and a 3 by 3 by $\frac{5}{16}$-in. angle was compared with a 5-in., 6½-lb. channel, the former intended for the anchor and cantilever arms and the latter for the suspended span. These test pieces were made of the exact length, from center to center of inside rivets, that the lattice bars would have in the finished bridge, and were riveted to shoes adapted to bear on the plattens of the testing machine, so as to bring the compressive stress on the test piece in the same line, as nearly as possible, as would occur in the work. Of course, these tests did not represent the conditions of stress to which lattice bars would be subjected in a member of the bridge, but they furnished comparative tests, as nearly as it was practical to make them. An excess of rivets was used in the connections, because the object was to test the sections and not the riveted connections. Table 2 gives the results of these tests.

TABLE 2.—Results of Tests of Channels and Angles.

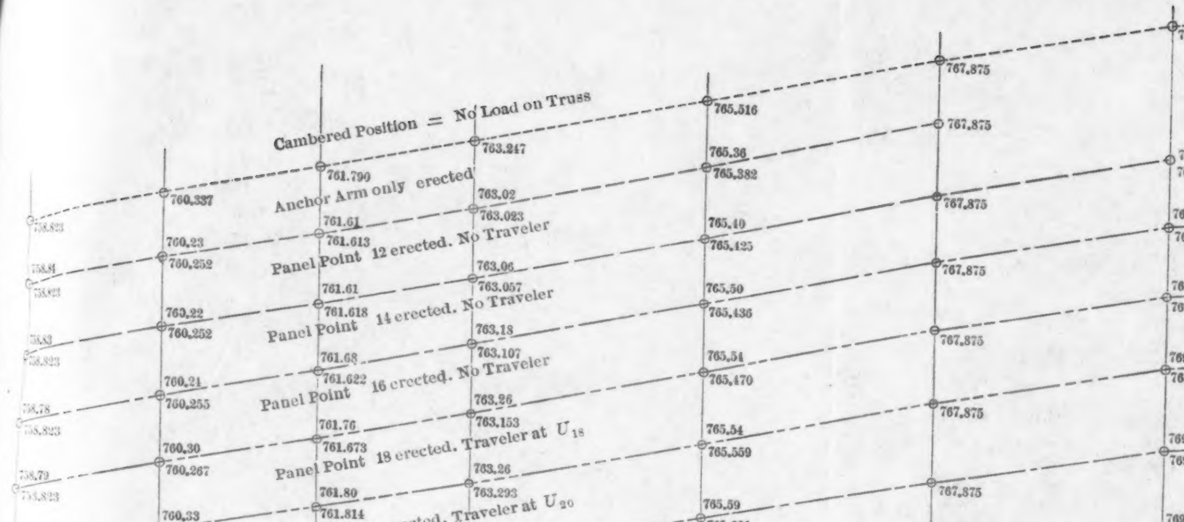
	Ultimate Strength.		
	Total.		Per square inch. actual. in pounds.
	Computed,* in pounds.	Actual, in pounds.	
6-in., 10-lb. channel	53 600	55 500	17 950
3½ by 3½ by 5/16-in. angle	47 750	47 050	22 820
5-in., 6½-lb. channel	32 200	30 450	15 620
3 by 3 by 5/16-in. angle	38 700	46 070	25 870

* These were computed by Tetmajer's formula for the ultimate strength of small columns, as given by C. C. Schneider, Past-President, Am. Soc. C. E., in his report on the Quebec Bridge, $f = 44\,100 - 162 \frac{l}{r}$, modified by the calculated effect of eccentricity of connections.

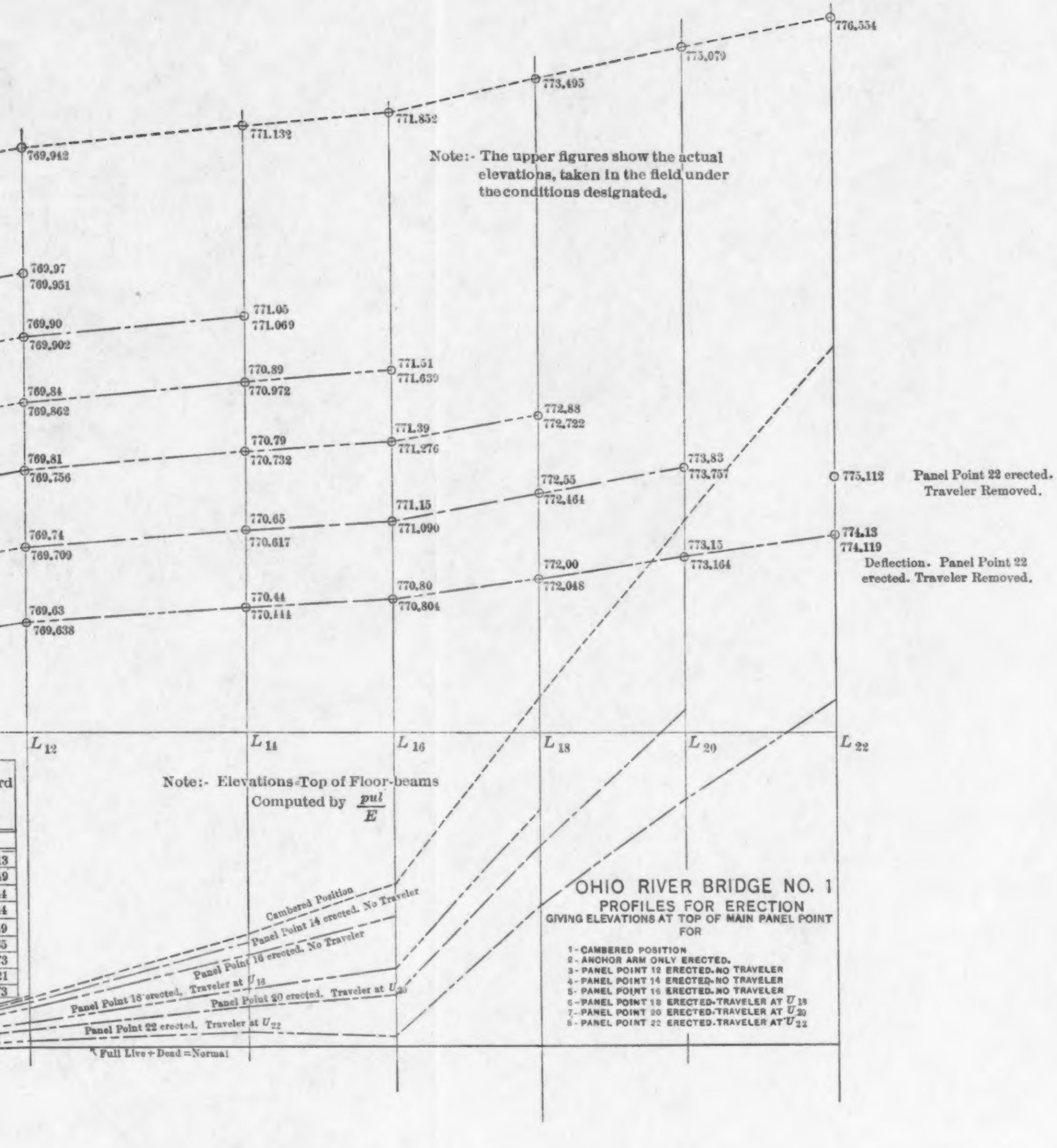
These tests indicate an efficiency for the angles of from 30 to 50% greater than for the channels, and the computations show that this is due to the very small eccentricity of the angle connections.

The deflections were recorded and the load removed with each increment of 5 000 lb. up to 30 000 lb., at which point the 6-in. channel showed a permanent set, whereas the 3½-in. angle did not. At 40 000 lb. the 6-in. channel showed a flexure 25% greater than the 3½-in. angle. The permanent set of the 5-in. channel and the 3-in. angle was not obtained. The flexure was always in the direction indicated by the slight eccentricity of the riveted connections, except for the 3-in. angle, which began to deflect in the opposite direction; but, after the load reached 40 000 lb., the eccentricity of the connection caused the flexure to reverse. Between the shops and the testing laboratory, this angle had been bent in handling or shipping; therefore it was a defective test piece at the start. The test, however, proved that this initial bend was in the direction which neutralized the effect of eccentricity until the load reached 40 000 lb., and it is probable that the ultimate load was greater than would have been the case if the angle had been straight. All these test pieces failed by column flexure, the back of the channels and the root of the angles being concave. On the basis of these tests, the angle sections were approved and used instead of the channels in the lower chords from L_0 to L_6, and from L_{14} to L_{22}. They afforded ample room for the riveted connections, and the saving in weight fully compensated for the additional cost of fabrication. It also avoided cutting and splicing at intersections, which is an objection to the channel lattice bar.

Shoes and Connections of Floor-Beams and Sidewalk Brackets at L_{10}. —The shoes at L_{10}, which carry the entire load that comes on the cantilever piers, are built up of plates and angles with three ribs and three diaphragms, and take their loads through 14-in. pins. They have a base of 8 by 9 ft., and a height, to center of pin, of 8 ft. 6⅛ in. The thickness of the bearing of the three ribs is 17¾ in., and the shipping weight is 37 860 lb. The shoes are connected across the pier by box struts, 4 ft. deep and 2 ft. 6 in. wide, latticed on four sides and riveted to the shoes. The end connection plates of these struts are extended up to form a seat for the floor-beam at L_{10}, bringing the end reactions of this beam directly to the shoes. This detail was adopted in order to avoid the almost impossible field connections of the floor-beam to the tower columns, as contemplated by the contract plans. The brackets support-



Center of Lower Chord Below Top of Floor-beams	
At	Feet
L_0	3.313
L_1 to L_5 incl.	3.349
L_6	3.344
L_7 to L_{11} incl.	3.344
L_{12}	3.349
L_{13} to L_{15} incl.	3.365
L_{16} to L_{21} incl.	3.573
L_{22}	3.521
L_{23}	3.573


776.554
775.079
773.495
771.852
771.132
769.942
Note:- The upper figures show the actual elevations, taken in the field under the conditions designated.
769.97
769.951
771.05
771.069
769.90
769.902
770.89
770.972
771.51
771.639
769.84
769.862
772.88
772.722
771.39
771.276
770.79
770.732
769.81
769.756
773.83
773.757
775.112
Panel Point 22 erected. Traveler Removed.
772.55
772.464
771.15
771.090
770.65
770.617
769.74
769.709
774.13
774.119
Deflection. Panel Point 22 erected. Traveler Removed.
773.15
773.164
772.00
772.048
770.80
770.804
770.44
770.444
769.63
769.638
L_{12}
L_{14}
L_{16}
L_{18}
L_{20}
L_{22}
Note:- Elevations-Top of Floor-beams Computed by $\frac{pul}{E}$
Cambered Position
Panel Point 14 erected. No Traveler
Panel Point 16 erected. No Traveler
Panel Point 18 erected. Traveler at U_{18}
Panel Point 20 erected. Traveler at U_{20}
Panel Point 22 erected. Traveler at U_{22}
Full Live + Dead = Normal
OHIO RIVER BRIDGE NO. 1
PROFILES FOR ERECTION
GIVING ELEVATIONS AT TOP OF MAIN PANEL POINT FOR
1 - CAMBERED POSITION
2 - ANCHOR ARM ONLY ERECTED.
3 - PANEL POINT 12 ERECTED. NO TRAVELER
4 - PANEL POINT 14 ERECTED. NO TRAVELER
5 - PANEL POINT 16 ERECTED. NO TRAVELER
6 - PANEL POINT 18 ERECTED - TRAVELER AT U_{18}
7 - PANEL POINT 20 ERECTED - TRAVELER AT U_{20}
8 - PANEL POINT 22 ERECTED - TRAVELER AT U_{22}

ing the sidewalk at the tower, L_{10}, were also supported directly from the shoes for the same reasons.

Forked Connections at Main Panel Points Common to Two Diagonal Compression Members.—To meet problems of packing, and to facilitate erection, the sub-diagonals, M_7-L_8 and L_8-M_9, were connected at L_8 on common pin-plates which formed a fork. This was shop-riveted to L_8-M_9 with field-riveted connections for M_7-L_8, 3 ft. 6 in. from the pin center. The same detail was used at L_{12} and L_{18}. The detail at L_{10}, for the members, U_8-M_9-L_{10} and L_{10}-M_{11}-U_{12}, is similar, except that the fork is a separate member with field-riveted connections for both of the main members.

This detail introduced some secondary stresses, which were duly considered and provided for. In this case these were neither excessive nor difficult to handle, but, with heavier members, the secondary stresses might be objectionably large.

Trussing in Center Cross Panels of Suspended Span.—The contract plans called for cross diagonal eye-bars, without adjustment, in the center panels of the suspended span, U_{22}-M_{23}-L_{22}. As in all other parts of the bridge the lower sub-diagonals were compression members, and as it was necessary to introduce adjustment in these panels, the upper sub-diagonals, U_{22}-M_{23}, were made of adjustable eye-bars connecting on the pin, M_{23}, outside of all the other members. The lower sub-diagonals on the north side (the first side erected) carried shop-riveted pin-plates at M_{23}, which plates were provided with field-riveted connections for L_{22}-M_{23} (south side), U_{23}-M_{23}, and M_{23}-L_{23}. This member was erected on the north side, and was held in position by the adjustable diagonals, U'_{23}-M_{23}, engaging the pin, M_{23}. In closing the bridge, all the other members connecting on M_{23} came together automatically and without any difficulty, except that required to guide them into the connection plates. As soon as the suspended span was swung, the upper adjustable diagonals on the south side were slipped over the ends of the pins and the nuts were then screwed on, completing the connection.

Connection of End Post to Top Chord of Suspended Span at U_{18}.—The end posts of the suspended span have, shop-riveted to their upper ends at U_{18}, and integral with them, a short section or stub end of the upper chord. The main chord section is field-riveted to this piece 3 ft. 7 in. forward of the pin, U_{18}. The short section of upper chord is extended back of the panel point 5 ft. 3 in. to take the subsidiary pin to

which the toggle bars connect. This detail permitted all the connections to be made on the pin, U_{18}, without the necessity of supporting a long section of upper chord projecting out. It also permitted the traveler to be erected with its principal reaction at U_{18} before connecting the upper chord section, U_{18}-U_{20}. This short section or stub end of upper chord is connected to the end post with heavy gusset-plates, sufficient to take care of all secondary stresses which occurred at this point during erection. The separate pin, U_{18} A, for the toggle bars, permitted the triangle, U_{16}-L_{16}-U_{18}, to be connected and made self-supporting before making the pin, U_{18}.

Provisions for Wind Shear in Bottom Frame.—Except at L_{16}, where there is a slip joint, the wind shear in the bottom frame is carried by the buckle-plates and their connections to the stringers and floor-beams, and is transmitted to the lower chord through the connections of the floor-beams to the vertical truss members and of the latter to the lower chord by pins at the main panel points and by riveted diaphragm connections at the sub-panel points. These connections were all investigated and found to be ample, except at the sub-panel points, L_7, L_9, L_{11}, and L_{13}, where the sub-verticals were somewhat deficient in transverse strength to transmit the load safely; but, as there is a considerable excess of strength at the main panel points on each side (L_6, L_8, L_{10}, and L_{12}), it was assumed that the stringers and their connections, particularly the lines adjacent to the trusses, would take up the excess shear at these sub-panel points and distribute it to the main panel points.

The wind shear at L_{16} is transmitted to the lower chords at L_{15} through a triangular frame which has its apex in a cross-head sliding between two brackets at the center of the floor-beam, L_{16}. This detail is not novel, but was worked out independently, before the description of a similar one had been published. The legs of the triangle are composed of four angles latticed in an I section. They are braced laterally, by a frame, to the bottom chord or erection strut just back of the wedge pin, serving to brace the latter during erection. This is shown on Fig. 5.

Erection.

General Plan.—Naturally, the erection of steel began at each side with the riveted pony truss spans, of which there were three, 75 ft. long,

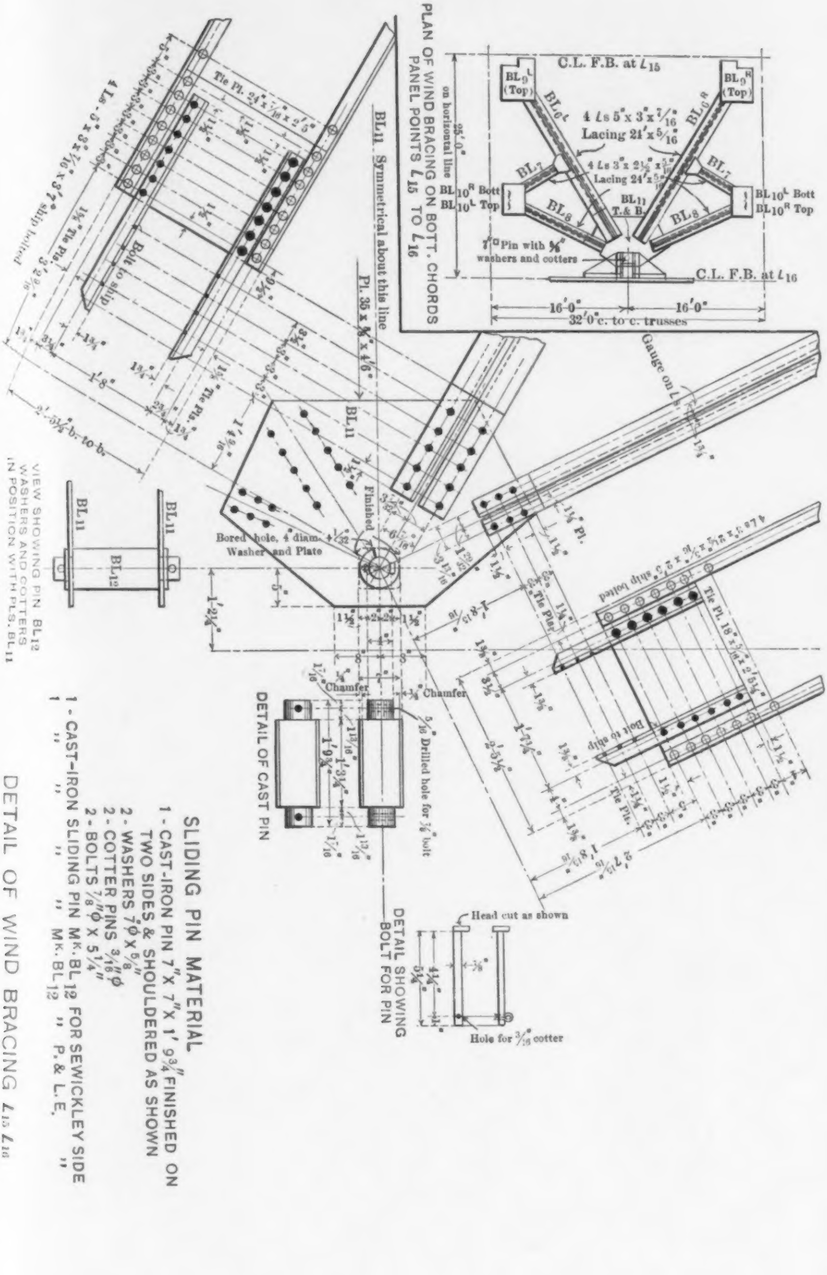
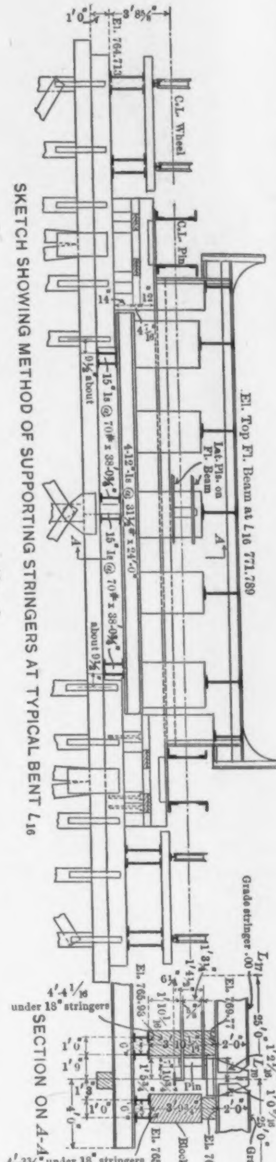


Fig. 5.

DETAIL OF WIND BRACING L_{15} L_{16}

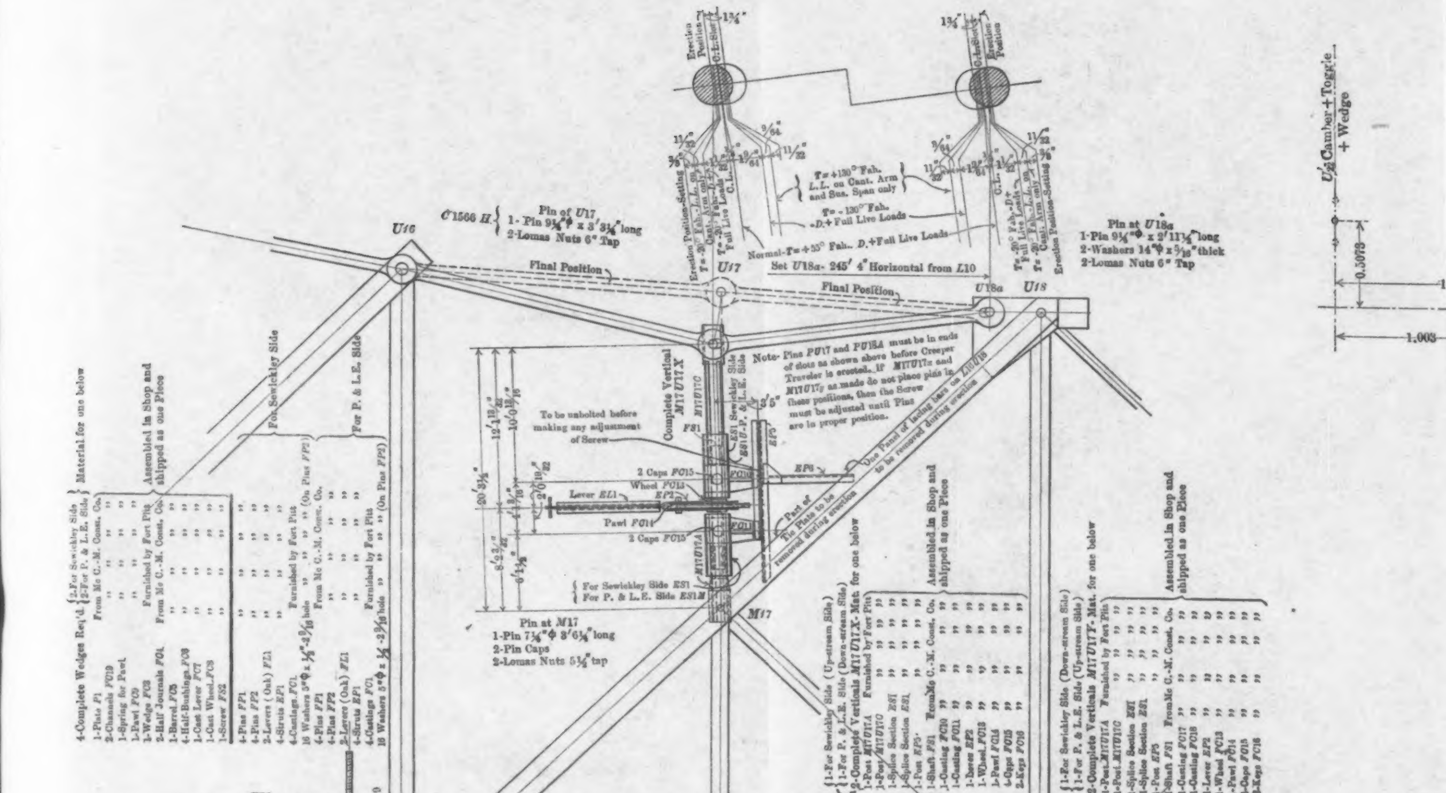
on the Sewickley or north side, and two of 75 ft. with one of 118 ft. over the Pittsburgh and Lake Erie tracks, on the south side. These trusses were assembled on the ground and lifted into place with the 30-ton locomotive crane. In setting the 118-ft. trusses, the locomotive crane was assisted by a boom derrick.

Falsework.—The falsework for the anchor arms was assembled in single stories and lifted into place by the locomotive crane, the piles having already been driven and cut off level. Plate LXXXVI shows the bents under panel points 7, 8, and 9, and Fig. 6 shows the provision for supporting the stringers and locomotive crane at panel point 16. As soon as the upper story on each bent was completed, the necessary blocking was arranged and the floor-beams and stringers were set thereon in final position and bolted up. The buckle-plates were then placed and bolted, and on them the track was laid for the advance of the locomotive crane. This operation was repeated, bent by bent, until the falsework reached the cantilever pier or one or two panels beyond. The wing or gantry traveler was then erected on the falsework, stock I-beams being used to support the traveler tracks on each side, as shown in the plans. As the erection of the steelwork for the anchor arm trusses proceeded, the locomotive crane continued to set the falsework bents and the floor-beams and stringers, out to panel point 18.

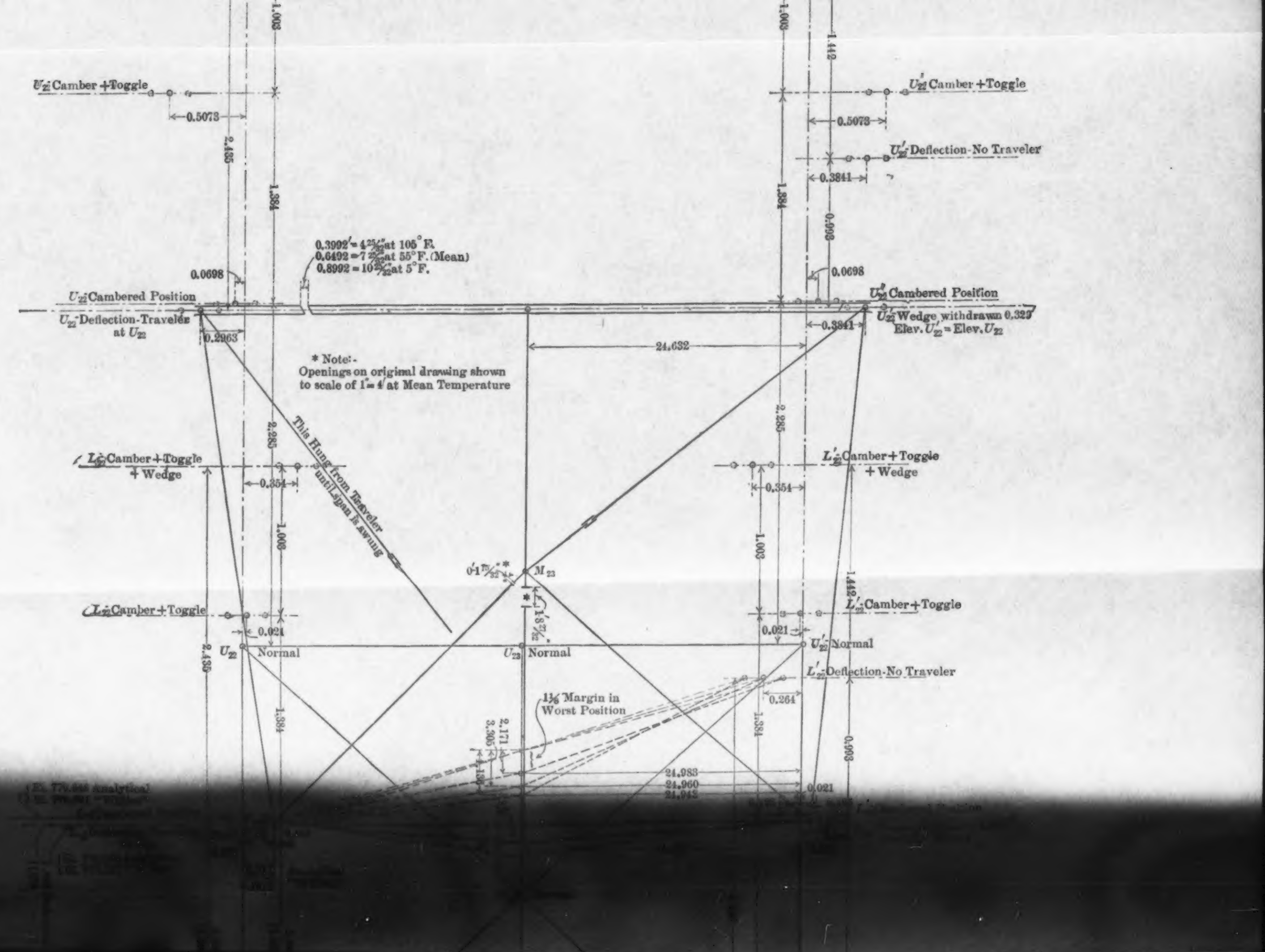


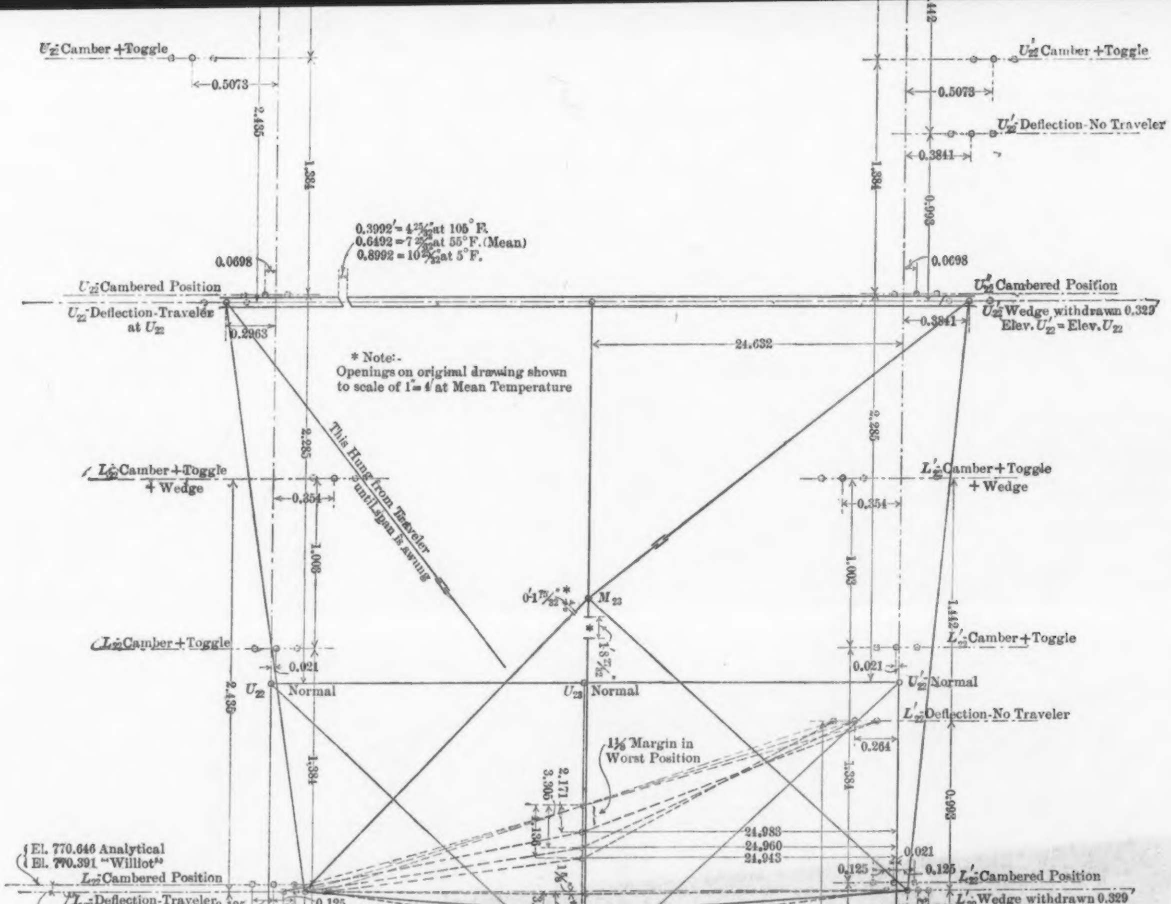
SKETCH SHOWING METHOD OF SUPPORTING STRINGERS AT TYPICAL BENT L_{16}

Fig. 6.



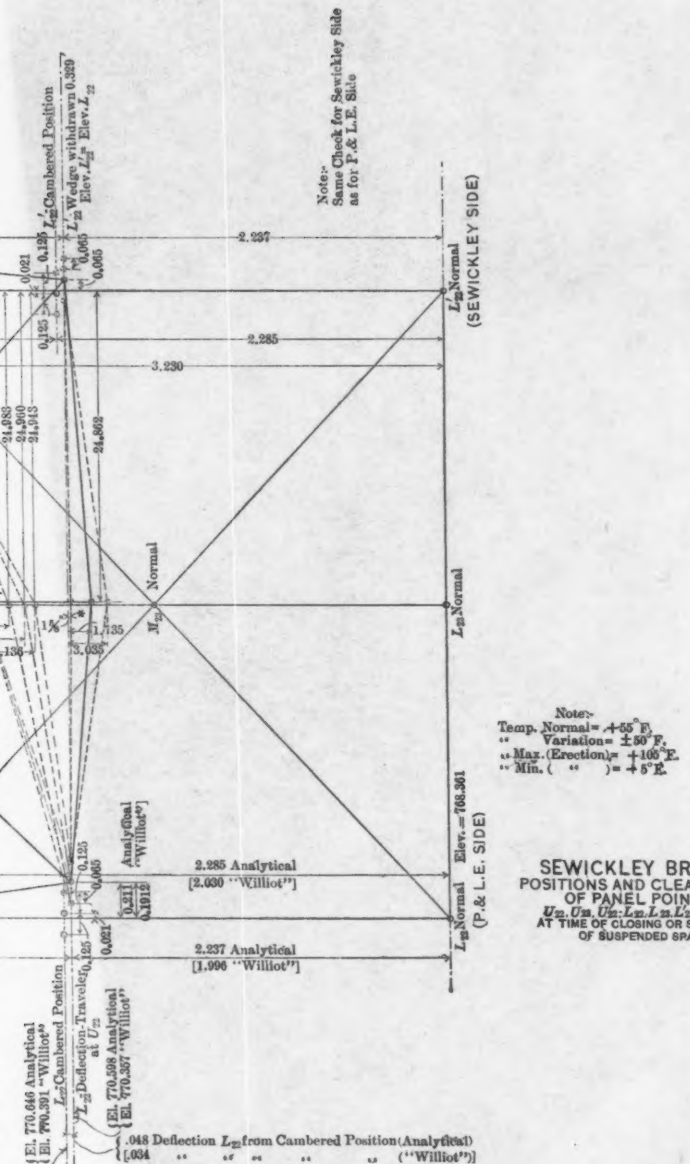
View of Inside of Truss
Down-stream P. & L.E.
Up-stream Sewickley
U16
U17
U18a
U18
M17
L16
L17
L18
FL1
FC19
FP1
FP2
FC1
Final Position
Pin of U17
1- Pin 9¼"φ x 3' 3¼" long
2-Lomas Nuts 6" Tap
C1566 H.
Pin at U18a
1-Pin 9¼"φ x 2' 11¼" long
2-Washers 14"φ x 3/16" thick
2-Lomas Nuts 6" Tap
Set U18a- 245' 4" Horizontal from L10
Normal-T= +55° Fah.. D.+Full Live Loads
T= +130° Fah. L.L. on Cant. Arm and Sus. Span only
T= -130° Fah. D.+Full Live Loads
Erection Position
Erection Position-Setting
Complete Vertical M17 U17 X
Note- Pins PU17 and PU18A must be in ends of slots as shown above before Creeper Traveler is erected. If M17U17x and M17U17y as made do not place pins in these positions, then the Screw must be adjusted until Pins are in proper position.
To be unbolted before making any adjustment of Screw
2 Caps FC15
Wheel FC13
Lever EL1
Pawl FC14
Part of Tie Plate to be removed during erection
One Panel of lacing bars on L16U18 to be removed during erection
For Sewickley Side ES1
For P. & L.E. Side ES1M
Pin at M17
1-Pin 7¼"φ 3' 6¼" long
2-Pin Caps
2-Lomas Nuts 5¼" tap
Pin at L16b
1-Pin 9¼"φ x 3' 4" long
2-Lomas Nuts 6" tap
To be removed after Suspended Span is swung
Wedge to be set in initial position Bottom flush with castings FC1
One Panel of angle lacing on L14L16 C1566D to be removed and 6" x 3½" x ⅝" Ls bolted to bottom of chord to support Shoes FC1 until Pins L16A and L16B are in place and Wedge is in initial position. 6" x 3½" x ⅝" for use during erection only.
Parallel to Chord L14-L15
Pin at L16a
1-Pin 9¼"φ x 2' 5¼" long
4-Washers 13½"φ x ⅝"
2-Lomas Nuts 6" tap
Pin at L16
1-Pin 10½"φ x 2' 7¼" long
1-Pin Rod 1¼"φ x 3' 2¾"
2-Pin Caps
4-Complete Wedges Req'd 2-For Sewickley Side 2-For P. & L.E. Side Material for one below
Assembled in Shop and shipped as one Piece
For Sewickley Side
For P. & L.E. Side
Furnished by Fort Pitt
From Mc C.-M. Const. Co.
1-For Sewickley Side (Up-stream Side)
1-For P. & L.E. Side (Down-stream Side)
2-Complete Verticals M17U17X- Mat. for one below
1-For Sewickley Side (Down-stream Side)
1-For P. & L.E. Side (Up-stream Side)
2-Complete Verticals M17U17Y- Mat. for one below
U'32 Camber + Toggle + Wedge
U32 Camber + Toggle + Wedge
0.5078
1.003


U″22 Camber + Toggle
U′22 Camber + Toggle
U′22 Deflection-No Traveler
0.3992′ = 4 25/32″ at 105° F.
0.6492 = 7 25/32″ at 55° F. (Mean)
0.8992 = 10 25/32″ at 5° F.
U″22 Cambered Position
U″22 Deflection-Traveler at U22
U′22 Cambered Position
U′22 Wedge withdrawn 0.323
Elev. U′22 = Elev. U22
* Note:-
Openings on original drawing shown to scale of 1″ = 4′ at Mean Temperature
This Hung from Traveler until L span is swung
L″22 Camber + Toggle + Wedge
L′22 Camber + Toggle + Wedge
L″22 Camber + Toggle
L′22 Camber + Toggle
M23
U22 Normal
U23 Normal
U′22 Normal
L′22 Deflection-No Traveler
1⅛″ Margin in Worst Position
24.632
24.983
24.960
24.943
2.171
3.305
0.5073
0.3841
0.0698
0.2963
0.354
0.021
0.264
0.993
1.003
1.384
1.442
2.285
2.435
SEWICKLEY BRIDGE
POSITIONS AND CLEARANCES OF PANEL POINTS
CANTILEVER BRIDGE



U_{22}-Camber + Toggle
0.5073
U'_{22} Camber + Toggle
U'_{22}-Deflection-No Traveler
0.3841
2.435
1.384
0.993
0.3992' = 4 25/32" at 105° F.
0.6492 = 7 25/32" at 55° F. (Mean)
0.8992 = 10 25/32" at 5° F.
0.0698
U_{22}-Cambered Position
U_{22}-Deflection-Traveler at U_{22}
0.2963
U'_{22} Cambered Position
U'_{22} Wedge withdrawn 0.329
Elev. U'_{22} = Elev. U_{22}
24.632
* Note:-
Openings on original drawing shown to scale of 1" = 4' at Mean Temperature
This Hung from Traveler until span is swung
2.285
L_{22}-Camber + Toggle + Wedge
L'_{22}-Camber + Toggle + Wedge
0.354
1.003
M_{23}
L_{22}-Camber + Toggle
L'_{22}-Camber + Toggle
1.442
0.021
U_{22} Normal
U_{23} Normal
U'_{22} Normal
L'_{22}-Deflection-No Traveler
0.264
1⅛" Margin in Worst Position
2.171
3.305
24.983
24.960
24.943
0.125
El. 770.646 Analytical
El. 770.391 "Williot"
L_{22}-Cambered Position
L'_{22}-Cambered Position
L'_{22}-Wedge withdrawn 0.329

PLATE LXXXVIII.
PAPERS, AM. SOC. C. E.
SEPTEMBER, 1912.
BUEL ON
THE SEWICKLEY CANTILEVER BRIDGE.



SEWICKLEY BRIDGE
POSITIONS AND CLEARANCES
OF PANEL POINTS
$U_{22}, U_{23}, U'_{22}; L_{22}, L_{23}, L'_{22}$ and M_{23}
AT TIME OF CLOSING OR SWINGING
OF SUSPENDED SPAN

The locomotive crane set the falsework bents, floor-beams, stringers, etc., quickly and economically, fully justifying the trouble and expense incurred to provide for it. The heavy reactions, which often came almost entirely on two wheels of one truck, were difficult to take care of, and at one time it appeared to be impracticable to make use of this method. That a solution was finally reached is largely due to the resourcefulness of the erection department of the Bridge Company. The extra counterbalance was removed from the rear end of the locomotive crane, and special outriggers were provided which distributed the loads over four lines of stringers. The stringer reactions at the floor-beams were taken by shelf-angles with supporting stiffeners designed for erection loads. These have been referred to under the heading, "Camber and Finished Lengths of Truss Members."

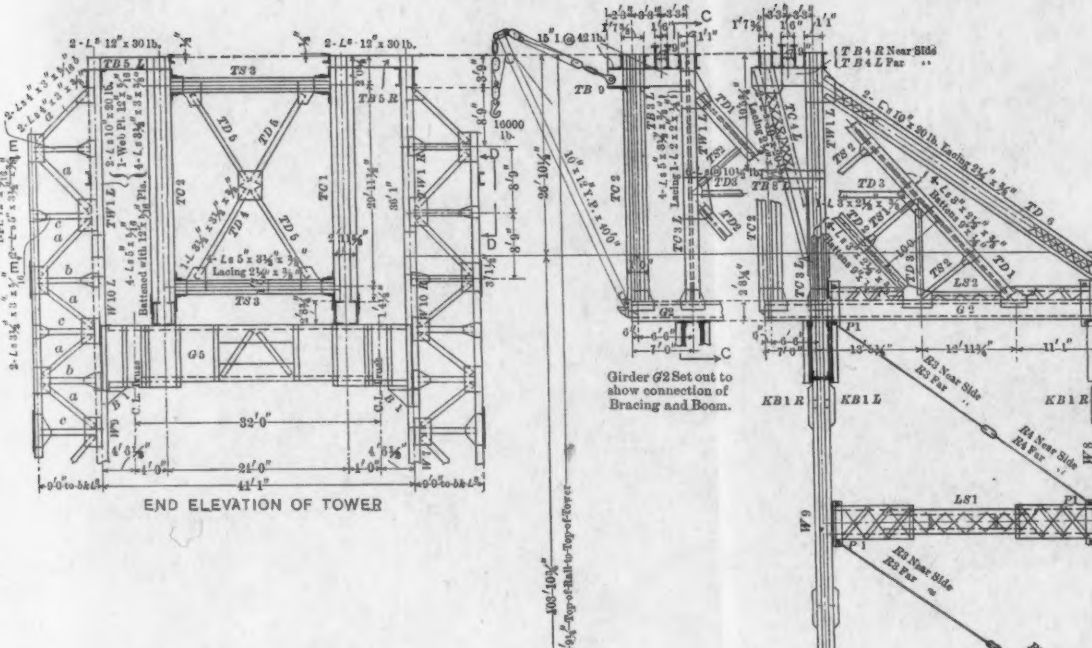
The original intention had been to erect the cantilever arm, from panel point 12 out, with a creeper traveler, and some provisions were made to do so; but, as the restrictions on account of navigation only required the maintenance of a clear waterway of 500 ft., it was found permissible to carry the falsework out to panel point 18 on one side of the river at a time. After all the falsework was removed from the north side, it was replaced, out to panel point 18, on the south side, thus preserving the 500-ft. opening at all times. This method of erection, with the wing traveler on falsework out to the second panel point of the suspended span (18) was very economical and expeditious, and avoided considerable difficulty and expense in carrying the creeper traveler down the eye-bar chain of the cantilever arm. It also was of great assistance in making the panel points 16, 17, and 18, which, on account of the large number of members necessary to hold up at one time, were among the most difficult to close.

This plan of erecting the cantilever arm with falsework required careful provision, in order to avoid any possibility of a concentrated reaction occurring at any of the bents outside of the cantilever pier, which might easily have been caused by an unequal settlement of the falsework bents, and would have crushed the bents or buckled some of the truss members, particularly in the sub-panels. If the cantilever arm had been carried entirely by the falsework during its erection, it would have also been a difficult and hazardous operation to swing it without dangerously over-loading some of the bents and truss members, as the computed deflection of panel point 18, with the erection completed

to that point, was about 0.8 ft. With panel point 22 erected, the computed deflection of panel point 18 was about 1.45 ft. Each truss was fully adequate to carry its own weight as a cantilever, together with the weight of the traveler at L_{22}, which, of course, they had to do in erecting the suspended span. Therefore, as each successive panel was erected by the gantry traveler, from panel point 10 to panel point 18, the wedges were backed out at all points outside of the cantilever pier so as to swing the span entirely. With every load added to the trusses great care was exercised to make sure that the wedges were not too tight, but they were kept in place for lateral stiffness only and to hold the falsework in line with the steel superstructure. From this it will be seen that the falsework under the cantilever arm, after the locomotive crane had set the floor system, never carried anything except the gantry traveler and the panel of steelwork being erected at any time, but not yet connected.

As indicated in the preceding description, the erection was carried forward from panel points 18 by the true cantilever method, with a cantilever or creeper traveler moving on the upper chords of the suspended span. This traveler was erected by the wing traveler at panel points 16, 17, and 18, and was supported over the toggle bars by special I-beam tracks.

As shown by Plate LXXX, the roadway grade on the anchor arms is 3% ascending, which was introduced in order to get the head-room under the channel span required by the Government regulations. The grade from panel point 13 outward is 0.8%, with a vertical curve between panel points 10 and 13. The wheel base of the traveler was 75 ft., with three trusses, and this required special arrangements to get it over the vertical curve without overloading the stringers of the traveler track or the falsework bents. This condition was mitigated by using in the traveler track a flatter and longer vertical curve than that of the roadway, and by arranging the longitudinal sway rods in the traveler so that by slackening one pair of rods the traveler would deflect and partly adjust itself to the curve. The traveler was sufficiently strong in all details to carry its entire weight on one pair of wheels at each side, but, to have made the stringers and bents safe for such a concentrated load would have cost too much, and the danger of derailment would have been increased. The deflection of the traveler in the direction indicated was somewhat restrained by the longitudinal

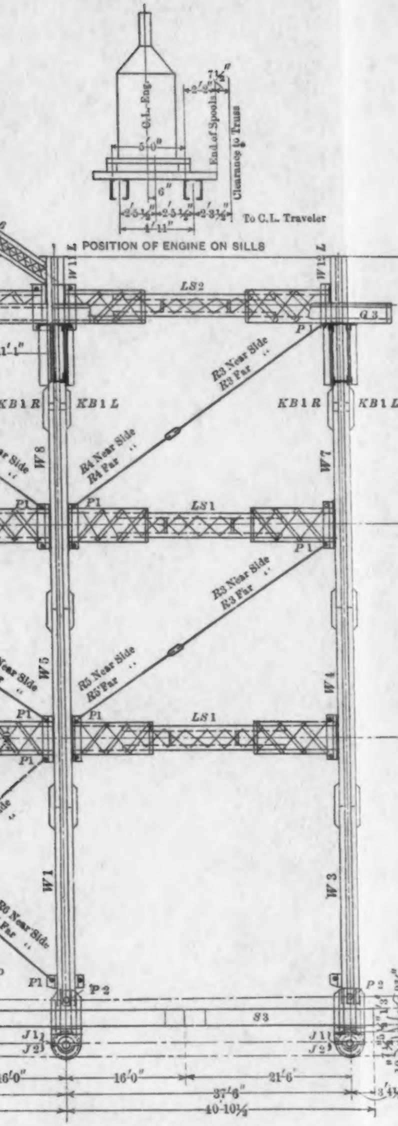


END ELEVATION OF TOWER

SILL PLAN

ELEVA

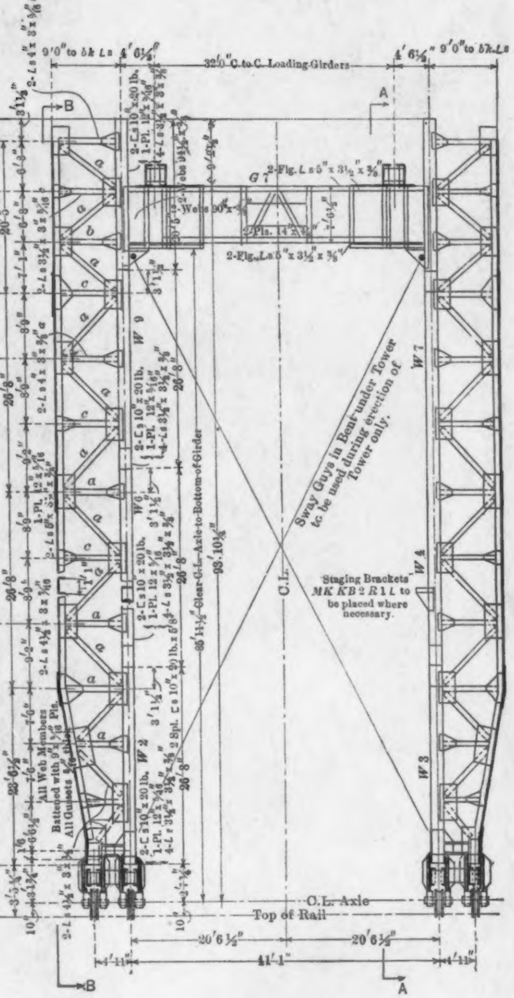
PLATE LXXXIX.
PAPERS, AM. SOC. C. E.
SEPTEMBER, 1912.
BUEL ON
THE SEWICKLEY CANTILEVER BRIDGE.



LEVATION A-A

ERECTION PLAN
GANTRY TRAVELER

Carrying Capacity, 50 Tons on each Wing Post.



struts, but a computation showed that the latter were not sufficiently stiff to cause undue concentrations when the sway rods were slack. These struts, however, proved to be so much stiffer than anticipated that they were used for staging in driving pins, etc., in preference to the staging brackets which had been provided.

The clear space between the top of rail of traveler track and bottom of traveler sill was 22 in. The largest pin that had to be driven through this space was 14 in. in diameter. This only allowed from 4 to 6 in. of clearance at the top and bottom to provide for variations in elevation and settlement of bents, camber blocking, etc. Several inches of this clearance were taken up by the adjustment of the vertical curves in the traveler track.

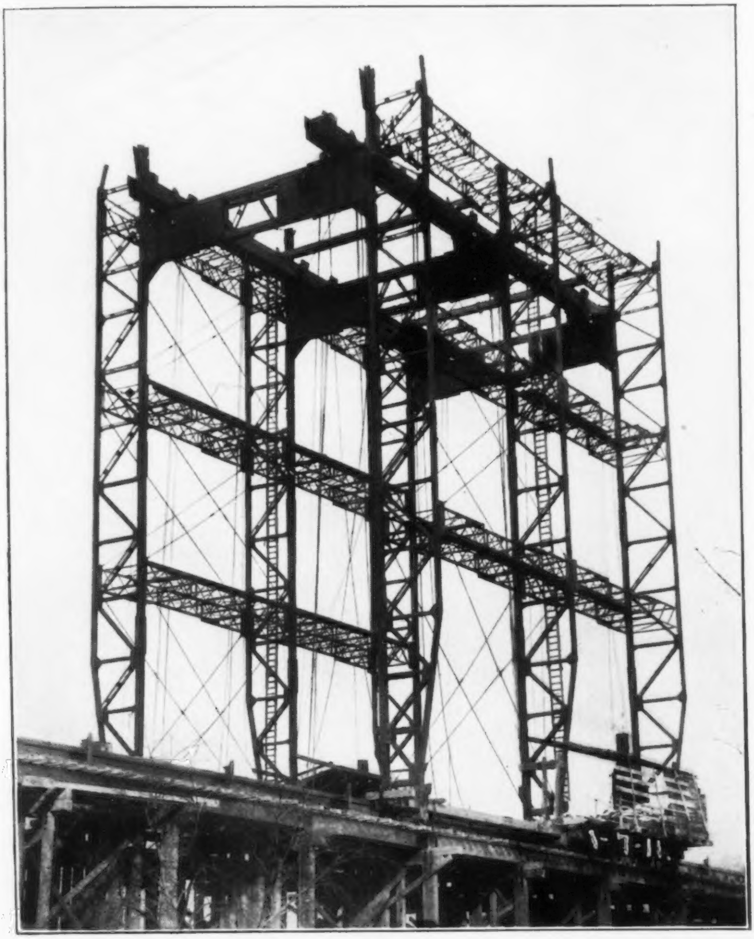
Much consideration was given to the hazard due to floods and other causes which might have destroyed the falsework. One of the provisions made for this was for swinging the anchor arm before the cantilever arm was erected. Erection ties were inserted, from U_2 to M_3 and from U_4 to M_5, and were removed as the shear in these panels was reversed, and the lower sub-diagonals were reinforced to meet this condition; but it was recognized that the greatest danger concerned the bents under the cantilever arms. These bents, 11 to 18, inclusive, did not carry any part of the steelwork after it had been fully connected, and therefore the hazard was practically confined to the possible loss of the falsework, provided only that the warning should afford sufficient time to run the traveler back to panel points 12, 13, and 14, lash it to the trusses, knock out the blocking, and let the falsework go. For floods and ice there would be sufficient warning for this. The trusses were more than able to carry this load except during the short time when panel point 18 had been made and the creeper traveler erected. With the two travelers on the cantilever arm at one time, there would have been some overloading, but this critical period was confined to the short time between the erection of the creeper traveler and the dismantling of the wing or gantry traveler. The work was planned so that this critical period was not encountered during ordinary times of high water. Fortunately, however, it did not become necessary to resort to any of these expedients.

Erection Deflections, Changes of Lengths, and Adjustments.—The deflections and elevations at the top of the floor-beams were computed analytically for the conditions of no load, for four conditions of loading in the finished bridge, and for seven conditions of erection loading.

Plate LXXXVII shows the deflection profiles for erection conditions. Williot graphical diagrams for displacements were also made for five conditions, the vertical co-ordinates being compared with and checked by the computed deflections. Plate LXXXVII shows the elevations taken on the floor-beams at different stages during the erection. The variations between the computed and actual deflections are generally quite small, and many of the differences would be very much less had it not been for the fact that some of the material, particularly for sidewalks and railings, was not erected in position but piled on the bridge at other points, whereas the computations considered all material as erected in final position, with no other material on the bridge excepting the traveler. The variations between the computed and actual deflections are less than might be expected when the differences in loading conditions are considered. The data from the Williot diagrams and from the computed deflections, Table 3, added to thermal changes, gave the minimum clearance or play required between U_{16} and U_{18} and between L_{15} and L_{16}, and also the run required for both toggles and wedges. On account of its extreme importance, these computations were made five times, by three different methods and by three men, independently. The provisions made are shown on Plate LXXXVIII.

At this point the question of toggles and wedges was taken up and carried so far as to determine the dimensions required for the principal parts and make a preliminary estimate of cost. Inquiries were then made as to whether or not the toggle and wedge apparatus from some previous cantilever bridge was available, with the result that the Bridge Company obtained the use of the set which had just done service in the Beaver Bridge. It had been designed for a load several times greater than the Sewickley Bridge, and required some special provisions to make it available, but there was the satisfaction of knowing that it had been tested under a much greater load than it would have to carry in this bridge. It is essentially the same as all the toggle and wedge devices that have been used in cantilever erections for the past ten years.

On May 15th, 1911, the lower chord of the suspended span was closed by driving the pins, L_{23}. The distance between U_{22} and U'_{22} was then measured and found to agree very closely with the computed clearance. The top chord was closed and the suspended span was swung on May 16th, 1911. During the entire work of erection, nothing of consequence



THE WING TRAVELER, SEWICKLEY BRIDGE.

TABLE 3.—DISPLACEMENTS AS DETERMINED BY WILLIOT DIAGRAMS, WITH PARALLEL COLUMNS SHOWING COMPARISON, OR CHECK, OF VERTICAL DISPLACEMENTS WITH VALUES COMPUTED BY $\frac{p\,u\,l}{E}$.

Point.	CAMBERED POSITION.					
	DISPLACEMENT.			CORRECTED CO-ORDINATES, ORIGIN AT L_{10}.		Corrected elevation above datum.
	Horizontal.	VERTICAL.		Horizontal.	Vertical.	
		Williot.	$\frac{p\,u\,l}{E}$			
L_{10}	0	0	0	0	0	764.531
L_8	— 0.0275	— 0.1060	— 0.1087	— 75.0275	— 2.3560	762.175
L_6	— 0.0560	— 0.1240	— 0.1282	— 150.0560	— 4.6240	759.907
L_4	— 0.0771	— 0.0865	— 0.0907	— 200.0771	— 6.0865	758.4445
L_2	— 0.0950	— 0.0460	— 0.0431	— 250.0950	— 7.5400	756.991
L_0	— 0.1030	— 0.0208 = $\frac{1}{4}$″ (adjusted)		— 300.1030	— 9.0208	755.5102
U_{10}	— 0.3464	+ 0.0583	....	— 0.3464	+ 114.7253	879.2563
U_8	— 0.1740	— 0.1300	....	— 75.1740	+ 77.7320	842.2630
U_6	— 0.1422	— 0.1012	....	— 150.1422	+ 53.9508	818.4818
U_4	— 0.1286	— 0.0684	....	— 200.1286	+ 45.9016	810.4326
U_2	— 0.1205	— 0.0270	....	— 250.1205	+ 42.4730	807.0040
U_9	— 0.2265	— 0.0920	....	— 37.7265	+ 94.5490	859.0800
M_9	— 0.0862	— 0.0655	....	— 37.5862	+ 38.8655	803.3965
L_9	....	....	....	— 37.5138	— 1.1780	763.3530
U_7	— 0.1520	— 0.1352	....	— 112.6520	+ 64.1948	828.7258
M_7	— 0.1150	— 0.1264	....	— 112.6150	+ 36.5546	801.0856
L_7	....	....	....	— 112.542	— 3.490	761.041
U_5	....	....	....	— 175.1354	+ 49.9262	814.4572
M_5	....	....	....	— 175.1078	+ 23.9326	788.4636
L_5	....	....	....	— 175.0665	— 5.3553	759.1757
U_3	....	....	....	— 225.1246	+ 44.1873	808.7183
M_3	....	....	....	— 225.1112	+ 19.1811	783.7121
L_3	....	....	....	— 225.0861	— 6.8132	757.7178
M_1	....	....	....	— 275.1114	+ 16.7269	781.2579
L_1	....	....	....	— 275.0990	— 8.2804	756.2506
L_{12}	+ 0.0223	+ 0.3534	+ 0.3600	+ 75.0223	+ 2.0554	766.5864
L_{14}	+ 0.0488	+ 0.7797	+ 0.7980	+ 150.0488	+ 3.2177	767.7487
L_{16}	— 0.2065	+ 1.0995	+ 1.1184	+ 199.7935	+ 3.7295	768.2605
U_{10}	— 0.3464	+ 0.0583	....	— 0.3464	+ 114.7253	879.2563
U_{12}	— 0.3024	+ 0.3298	....	+ 74.6976	+ 82.0318	846.5628
U_{14}	— 0.2235	+ 0.7840	....	+ 149.7765	+ 61.1540	825.6850
U_{16}	— 0.2065	+ 1.0761	....	+ 199.7935	+ 53.6901	818.2211
M_{10}	— 0.1480	+ 0.0247	....	— 0.1480	+ 49.0247	813.5557
U_{11}	— 0.3440	+ 0.1427	....	+ 37.1560	+ 96.8757	861.4067
M_{11}	— 0.1520	+ 0.1620	....	+ 37.3480	+ 41.0130	805.5440
L_{11}	....	....	....	+ 37.5115	+ 1.164	765.695
U_{13}	— 0.2560	+ 0.5504	....	+ 112.2440	+ 70.1314	834.6624
M_{13}	— 0.1262	+ 0.5545	....	+ 112.3738	+ 42.6245	807.1555
L_{13}	....	....	....	+ 112.5359	+ 2.7029	767.2339
U_{15}	....	....	....	+ 174.7865	+ 57.4176	821.9486
M_{15}	....	....	....	+ 174.9224	+ 28.4506	792.9816
L_{15}	....	....	....	+ 175.0567	+ 3.5759	768.1069
	DEAD LOAD ONLY. BRIDGE SWUNG.					
L_{10}	....	0	0	0	0	764.531
L_8	....	+ 0.0817	+ 0.0832	....	....	762.2567
L_6	....	+ 0.0970	+ 0.1007	....	....	760.004
L_4	....	+ 0.0583	+ 0.0614	....	....	758.5028
L_2	....	+ 0.0290	+ 0.0297	....	....	757.0200
L_0	+ 0.0750	+ 0.0205	+ 0.0205	....	....	755.5307
U_{10}	+ 0.2710	— 0.043	....	....	....	879.2133
U_2	+ 0.0976	+ 0.0179	....	....	....	807.0219
L_{12}	....	— 0.2816	— 0 2863	....	....	766.3048
L_{14}	....	— 0.6130	— 0.6181	....	....	767.1357
L_{16}	+ 0.1690	— 0.8615	— 0.8590	....	....	767.3990
U_{16}	+ 0.1690	— 0.8443	....	....	....	817.3768

TABLE 3.—(*Continued.*)

LIVE LOAD ON ANCHOR ARM ONLY.

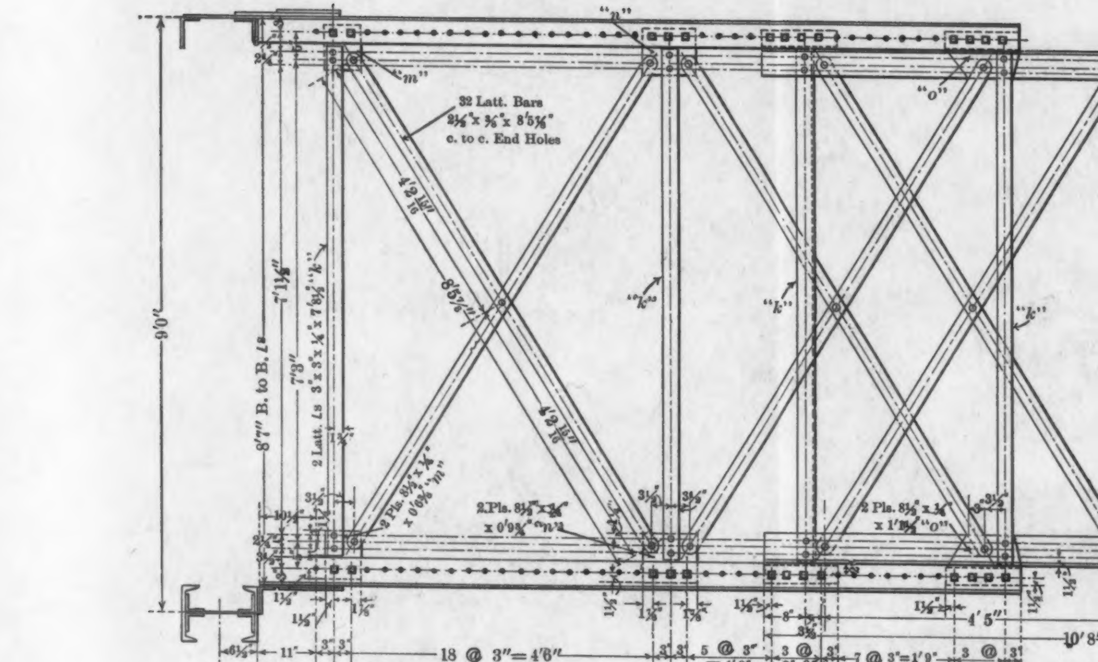
Point.	Displacement.			Corrected Co-ordinates, Origin at L_{10}.		Corrected elevation above datum.
	Horizontal.	Vertical.		Horizontal.	Vertical.	
		Williot.	$\frac{pul}{E}$			
L_{10}	0	0	0	0	0	764.531
L_8	....	+ 0.0235	+ 0.0234	....	....	762.1985
L_6	....	+ 0.0075	+ 0.0077	....	....	759.9145
L_4	....	− 0.0255	− 0.0251	....	....	758.4190
L_2	....	− 0.0396	− 0.0392	....	....	756.9514
L_0	....	0	0	....	....	755.5102
U_{10}	+ 0.2130	− 0.0428	....	....	....	879.2135
L_{12}	....	− 0.2368	− 0.2374	....	....	766.3496
L_{14}	....	− 0.5280	− 0.5342	....	....	767.2207
L_{16}	+ 0.1362	− 0.7460	− 0.7661	....	....	767.5145
U_{16}	+ 0.1362	− 0.7288	....	....	....	817.4923
LIVE LOAD ON CANTILEVER ARM AND SUSPENDED SPAN ONLY.						
L_{10}	0	0	0	0	0	764.531
L_8	....	+ 0.1580	+ 0.1596	....	....	762.3330
L_6	....	+ 0.2030	+ 0.2048	....	....	760.1100
L_4	....	+ 0.1667	+ 0.1686	....	....	758.6112
L_2	....	+ 0.0970	+ 0.0972	....	....	757.0880
L_0	....	+ 0.0350	+ 0.0350	....	....	755.5452
U_{10}	+ 0.3990	− 0.056	....	....	....	879.2003
L_{12}	....	− 0.3892	− 0.3917	....	....	766.1972
L_{14}	....	− 0.8505	− 0.8522	....	....	766.8982
L_{16}	+ 0.2322	− 1.1800	− 1.2021	....	....	767.0805
U_{16}	+ 0.2322	− 1.1566	....	....	....	817.0645
ANCHOR ARM ONLY SWUNG.						
L_{10}	0	0	0	0	0	764.531
L_8	....	− 0.1342	− 0.1341	....	....	762.0408
L_6	....	− 0.2234	− 0.2244	....	....	759.6836
L_4	....	− 0.1745	− 0.1767	....	....	758.2700
L_2	....	− 0.0851	− 0.0853	....	....	756.9059
L_0	....	0	0	....	....	755.5102
U_{10}	− 0.1339*	− 0.0010*	....	−0.4794′ = $-5\frac{3}{4}$*	+114.7243*	879.2553*
U_8	− 0.0772	− 0.1100	....	− 75.2512	77.6220	842.1530
U_2	+ 0.0028	− 0.0782	....	−250.1177	42.3948	806.9258
L_{12}	....	....	....	+ 75.0205*	+ 2.1417*	766.6727*
U_{12}	....	....	....	+ 74.6032*	+ 82.1174*	846.6484*

*Computed from Williot co-ordinates of U_8.—Members between U_8—L_{10}—U_{12} assumed at finished cambered lengths without stress.

occurred which had not been foreseen and provided for. There were no losses of either men or material, and no serious injuries were reported. One man fell into the water, but was rescued without serious results.

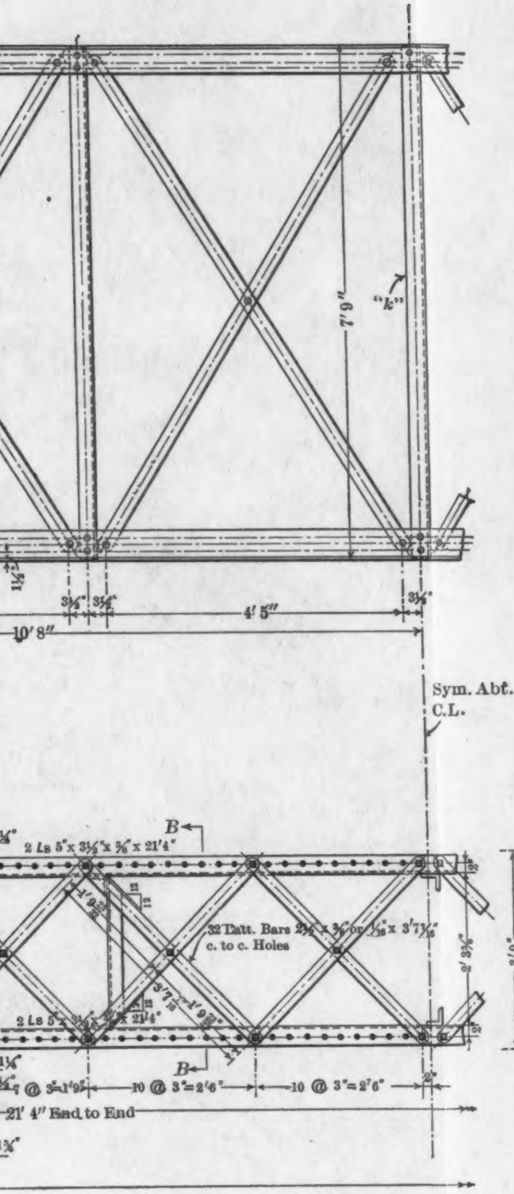
General Directions for Erection, and Order of Closing Operations Issued to the Erection Department on May 19th, 1910.—The spaces for pilot nuts, L_{18}, L_8 to L_{12}, etc., are somewhat scant. They had better be started off before the pin is entirely home, that is, $\frac{3}{8}$ in. back of the final position, or just flush.

The bottom chord splices of the anchor arm must be bolted and drifted before the anchor arm is swung.



32 Latt. Bars
c. to c. End Holes
9'0"
8'7" B. to B. Ls
18 @ 3"=4'6"
11'5"
SECTION "AA"
Near Side
For Pl. for Side only
Pl. for Side only
2 Ls 5"x 3½"x ⅜"x 11'4½"
12 Latt. Bars
c. to c. Holes
3'3"
11'5" Near Side
37'6" C.L. to C.L. Cols

PLATE XCI.
PAPERS, AM. SOC. C. E.
SEPTEMBER, 1912.
BUEL ON
THE SEWICKLEY CANTILEVER BRIDGE.



DETAIL OF LONGITUDINAL STRUTS GANTRY TRAVELER

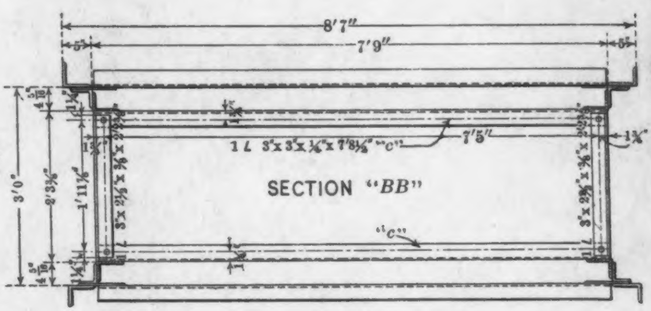
NOTE:-

Moment of Inertia	= 2598.0	Hor. Axis
Radius of Gyration	= 14.6	
Section Modulus	= 153.0	

Remove Lattice where clips interfere in adjusting to different Panel lengths, and replace lattice over clips if possible. If not practical to replace over clips, omit the panel of Lattice as, at clip, support is supplied by other section.

In adjusting to different panel lengths filler "g" under clip angles may have to be cut or else left projecting where lattice gussets come under clip angles.

Bolts to be used where indicated thus ◘
All other parts to be riveted



The expansion pockets for stringers on beam L_0 are not heavy enough to take the weight of the locomotive crane, and the stringers must be blocked up from the pier.

The eccentric and pin at P_1 must be well greased and covered with canvas to prevent rusting before the adjustment is completed.

Fully bolt and drift the top chord splices at U_{18} before advancing the creeper traveler beyond that point.

The locomotive crane must not be run out past the tower, L_{10}, after the creeper traveler is erected.

While the traveler is at U'_{22} (north side), and before moving it, connect the counters, U'_{22}-M_{23}, and the lower sub-diagonals, M_{23}-L'_{22} (which have shop-riveted gusset-plates at M_{23}), on pin M_{23}.

Be sure that the counters, U'_{22}-M_{23} (north side), are screwed up to just the right length, and that they are not too short in any case. If they are $\frac{1}{16}$ to $\frac{1}{8}$ in. too long, it will be better than to have them too short, because, if too short, L_{22}-M_{23} (south side) cannot be connected in closing.

With the traveler at U_{22} (south side), make the pin, L_{23}, engaging L_{22}-L_{23}, L_{23}-L'_{22}, and M_{23}-L_{23}.

The lower sub-vertical, M_{23}-L_{23}, having a field-riveted connection at M_{23}, will have to be tied up or supported until the span is swung.

Bring U'_{22} level with U_{22} by backing out the wedges on the north side at L'_{16}.

Set the upper sub-vertical, U_{23}-M_{23}, bolting it to the gussets at M_{23}.

Set the top chord, U'_{22}-U_{22}, bolting up the splice to U'_{22}-U'_{20} and to the top of U_{23}-M_{23}. It will then be self-supporting, its half-length, U_{23}-U_{22}, projecting toward the south side as a cantilever.

Set and bolt the top lateral struts, U_{23}-U_{23}, and the top laterals, U'_{22}-U_{23}.

Release the toggles until the top chord, U'_{22}-U_{22}, is in bearing at U_{22}, and bolt the splice.

The top laterals, U_{22}-U_{23}, must be put in place as soon as possible after the top chords, U_{22}-U'_{22}, are set, and before the toggles are fully released. When fully released, the span is changed from cantilever to span with both ends supported, and the conditions of the lateral frame change. With the traveler on top, this is serious, and it is very important to place the laterals as directed above.

After the top chord, U_{22}-U'_{22}, is set, and the center panel top laterals are in place, and the top chord splices are bolted (with toggles partly released), it is very important to continue slacking off the toggle screws until the toggle bars are in a straight line (*i. e.,* entirely slacked off), before leaving the bridge, because, in the position above described, a drop in the temperature would cause the top chords to act as suspension cables and probably cause a disaster.

The operation of slacking off the toggle screws may continue uninterruptedly after the top chord is connected.

The wedges should then be entirely slacked off before bolting M_{23}-L_{23} on the gusset at M_{23}.

After the span is swung, it will be necessary to lift the weight of the members on the pin, L_{23}, to take the sag out of the lower chord at this point before L_{23}-M_{23} can be bolted at M_{23}.

The counters, U_{22}-M_{23} (south side), can be slacked with turnbuckles and slipped over the ends of the pin, M_{23}, as these counters are outside (next to the pin nuts).

NOTE.—Prime (′) refers to the north side.

The Wing or Gantry Traveler.—An investigation having shown the impracticability of rebuilding the steel traveler which the Bridge Company had used during the previous year, so as to adapt it for the Sewickley Bridge, the following question was proposed: "Is it practicable to design a traveler adapted for the erection of spans of variable dimensions and weights?" Before the prices of materials and labor had advanced to their present values, and when good bridge timber was plentiful and cheap, it was common practice to build a special timber traveler for each job; but, in recent years, it has been found more economical to build travelers of steel, and they have generally been made without a sufficient range of adjustment to fit widely varying cases, although, in several instances, they have been designed with some adjustable members.

The foregoing question was answered by the proposal that, "a steel traveler is a tool or machine which should be provided with such adjustments as to make it available for the erection of all spans within a comprehensive range." A little preliminary work demonstrated the fact that it was entirely practicable to construct such a tool, and the result is the traveler first used on the Sewickley Bridge. The Bridge Company had a considerable quantity of stock which it was desired to use for the travelers. Some of this stock was perfectly good material, but of unusual sections rarely called for, and some of it, while good enough for a traveler, would hardly be acceptable for contract work. It was also desired that the material from the old steel traveler should be used in the new one, as far as available. These considerations required a number of unusual adaptations of details, but did not affect the general design materially.

Plate LXXXIX shows the general elevations of this traveler, together with the extension on top, designed for the erection of the

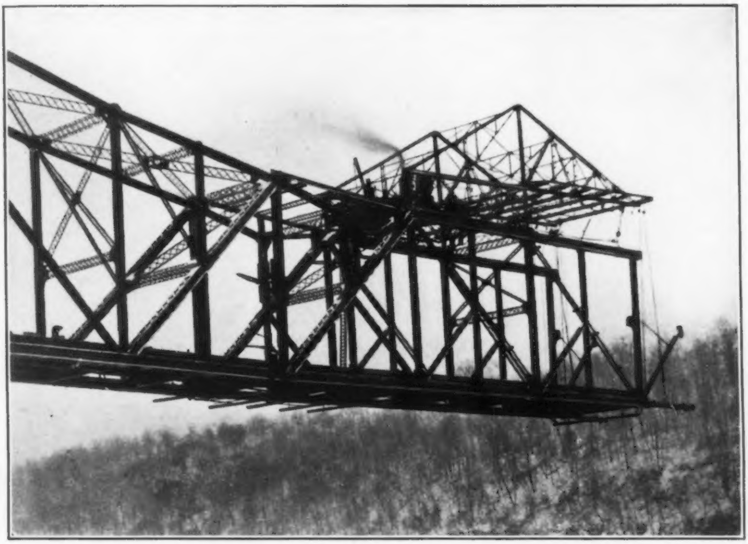


FIG. 1.—CREEPER TRAVELER AT U_{20}, NORTH SIDE.

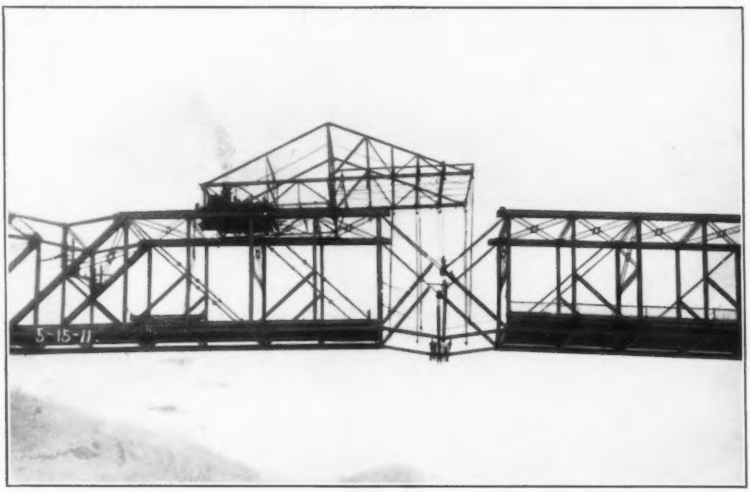


FIG. 2.—PINS AT L_{23} DRIVEN. CREEPER TRAVELER IN ADVANCE OF PANEL POINT 21.

towers. Fig. 7 shows the top view and plan of loading girders with and without tower. Plate XCI shows the details of the adjustable longitudinal struts.

The shoes which carry the boxes of the main wheel journals and the pins connecting the lower ends of the wings can be adjusted for panel lengths varying by 6 in. For the Sewickley Bridge, it was bolted up for panels 37 ft. 6 in. long. The maximum reach is two 40-ft., and the minimum two 24-ft. panels. For two 32-ft. panels, one of the 16-ft. sections of sill would be removed and the center shoe shifted to the center of the remaining 16-ft. section. For two 24-ft. panels, the other 16-ft. section would be removed. The projection of the sills beyond the ends is thus kept within 8 ft. 3 in. The traveler may be used with two trusses or wings instead of three.

The wings were designed so that when bolted up in a total length of about 100 ft., and lying down flat, they can be supported on one point at their center. This provision was made so that they might be tripped up from the bottom pin with a tackle made fast to one point.

The longitudinal spacing in the wings is uniformly 5 in. from end to end, with 2½ in. edge distance at the ends of the sections, and bolts are used instead of rivets throughout all sections where the transverse girders might come. This permits the height from traveler rail to transverse girders to be adjusted to any distance up to about 95 ft., by variations of 5 in. The capacity of the traveler is sufficient to permit the addition of other sections of wings, so that the clear height could be extended up to about 125 ft.

The transverse box girders have open lattice in their center 10-ft. panels and solid-plate webs in the end panels. The latter are engaged by another set of web-plates with separate flange-angles, arranged to bolt on the outside of the center section. The holes in both sets of web-plates are punched, so that they can be bolted in any position to give clear widths of from 30 ft. 3 in. to 49 ft. 3 in. between the traveler wings. For the Sewickley Bridge, they were set at 40 ft. 3 in. clear. It is intended that these adjustments shall be made at the shop for each job, before shipping the traveler.

The loading girders were designed to give a large moment of inertia sideways. Their make-up was determined by the stock on hand. Open holes were provided in the webs to fix the saddles and prevent them from sliding longitudinally when drifting a load. Open holes were also

provided in the end diaphragms for bolting on brackets to carry Chicago boom seats. These booms were used to set the top lateral system between U_8 and U_{12}, after the traveler had moved forward, and also to assist in supporting one end of the upper chord, U_{10}-U_{11}. The loading girders were riveted up complete at the shop, but it proved so difficult to handle them that, before taking down the traveler on the north side, the rivets connecting them through the diaphragms were cut out and bolts were substituted when they were set up again on the south side. This reduced the weight necessary to handle at one time to about one-half.

The most difficult adjustment to work out was that of the longitudinal struts (Plate XCI). The center section of these struts, which is 2 ft. 3⅜ in. deep and 7 ft. 9 in. wide, back to back of angles, telescopes into the two end sections, which are 3 ft. deep and 8 ft. 7 in. wide, back to back of angles, eight clip angles, 3½ by 3½ by ⅜ in. and 13½ in. long, at each end, providing a good long connection and 1 in. of clearance between the sections. In making this adjustment, which will generally be done at the shops, the clip angles will be removed and replaced in the required positions.

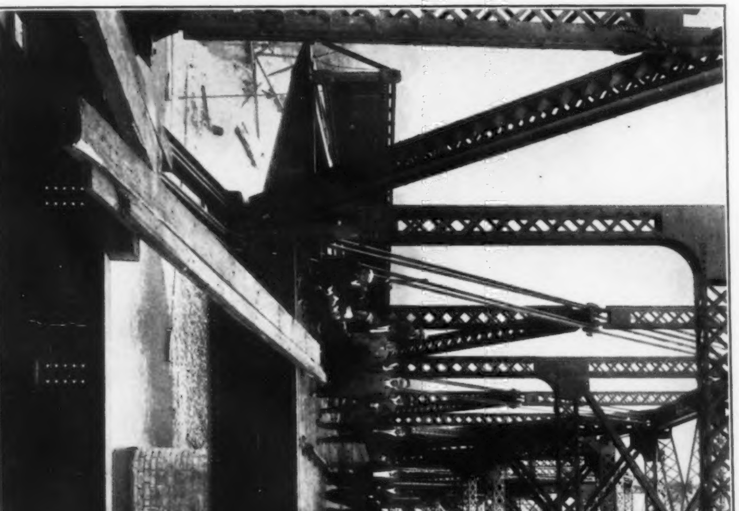
The only parts of this traveler which are not adjustable for spans of different dimensions are the 1½-in. round lateral and longitudinal sway rods. It was considered preferable to avoid complications incident to introducing adjustments of wide range in these rods because it is very simple to make new pieces for the short ends, and the discarded pieces will be good stock which can be readily reworked.

The eight saddles for the main hoists were designed so as to travel on the loading girders with four wheels on two axles, and two of them were thus constructed. The other six were constructed with cast shoes and two 3½-in. pins to transmit the load to the girders. These pins are identical with the axles of the two fitted with wheels, so that they can be used either with wheels or with stationary shoes.

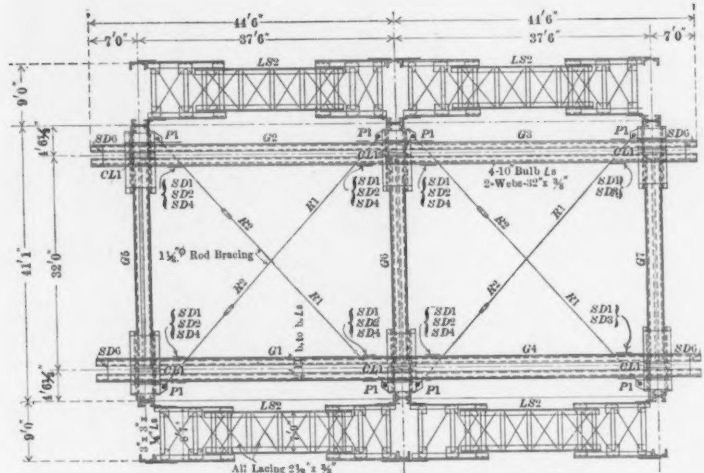
The twelve main traveler wheels were made with cast-iron centers or treads and double steel-plate flanges, riveted through, and with 7-in. journals, calculated to carry the maximum load when lifting. These wheels gave good service and no trouble. The two Mundy hoisting engines, one on each side, had three drums and four spools, and were set up on the sills in the forward bay, the longitudinal sway-rods being omitted in this bay. It was intended to use blocking between the sills

CROSS-SECTION OF SEWICKLEY BRIDGE.

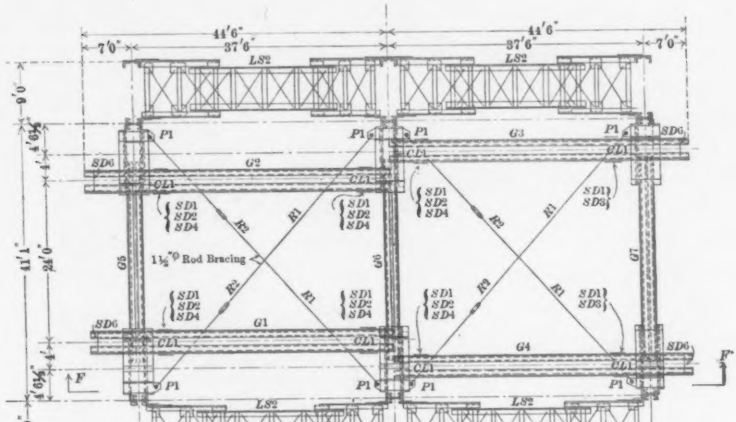
PLATE XCIII.
PAPERS, AM. SOC. C. E.
SEPTEMBER, 1912.
BUEL ON
THE SEWICKLEY CANTILEVER BRIDGE.



CROSS-SECTION OF SEWICKLEY BRIDGE.



TOP VIEW OF TRAVELER(WITHOUT TOWER)



PLAN OF LOADING GIRDERS. TOWER ERECTED.

GANTRY TRAVELER

FIG. 7.

and the traveler track under the engines, because the sills were not sufficiently stiff to carry them when working. In place of the blocking, an extra pair of smaller wheels was afterward placed under the engines on each side. This improvement was made by the erection department.

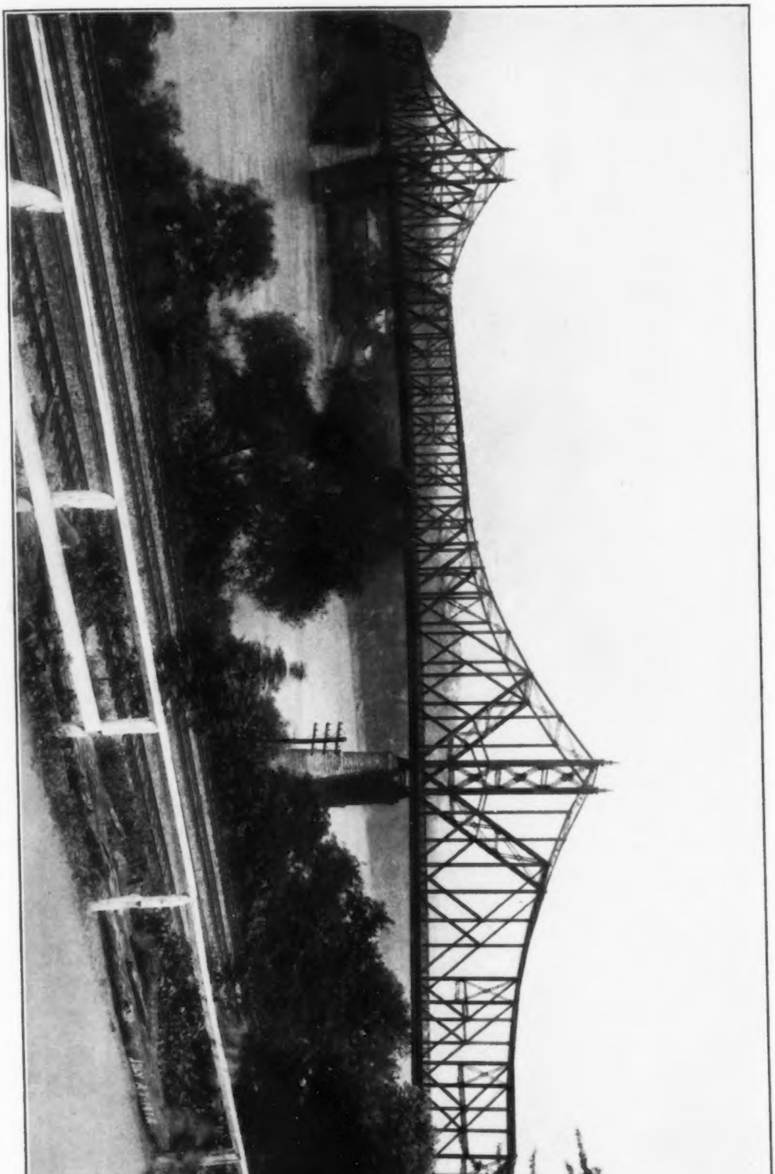
The traveler, as set up for the Sewickley Bridge, cleared the upper chord panel points, 8 and 12, but was not high enough to reach the pins, U_9, U_{10}, and U_{11}. The Bridge Company wished to avoid, if possible, the risk incident to rigging boom derricks at the elevation ot the top of the traveler. The lifts at this point were quite heavy, the lower sections of the tower weighing more than 34 tons and the upper sections more than 31 tons each. The eye-bars for each panel of the upper chord at the tower weighed more than 13 tons. To meet this part of the problem, an extension was built on the rear end of the traveler, as illustrated on Plate LXXXIX and Fig 7. The loading girders were shifted toward the center to clear the trusses, and carried, on their 7-ft. outlook or overhang, a pair of columns braced back to the girders, which, with an extension section on the rear wing braced back to the center wing and a batter post from the same, formed a gallows frame centered over the trusses. Extensions of the beams forming the top of this gallows frame provided attachments for light hoists to handle the pins, driving rams, etc. Supports for scaffold platforms were also arranged at a convenient height. The efficiency of this device met all expectations.

This traveler was designed for loads of 50 tons at each of the six wings at one time, in addition to its own weight and equipment. The saddles for the main hoist were also designed for loads of 50 tons. The loading girders are good for 23.3 tons at the center of a 37.5-ft. span, and for more than 50 tons on the outlook at a point 4 ft. from the center of support. The heaviest piece in the Sewickley Bridge was the lower section of the towers which weighed a little more than 34 tons.

In designing the loading girders and main hoist saddles, some consideration was given to the possibility of future electric equipment, and it is thought that there will be no difficulty in installing electric hoists on top of the main hoist saddles and, for the lighter hoists, at other points on the upper deck. With electric equipment, a controller cage, located so as to give the best field of view, would be installed in the wings.

Creeper Traveler.—Fig. 8 shows the stress, section, and loading

PLATE XCIV.
PAPERS, AM. SOC. C. E.
SEPTEMBER, 1912.
BUEL ON
THE SEWICKLEY CANTILEVER BRIDGE.



THE SEWICKLEY BRIDGE OVER THE OHIO RIVER. NEAR VIEW.

FIG. 8.

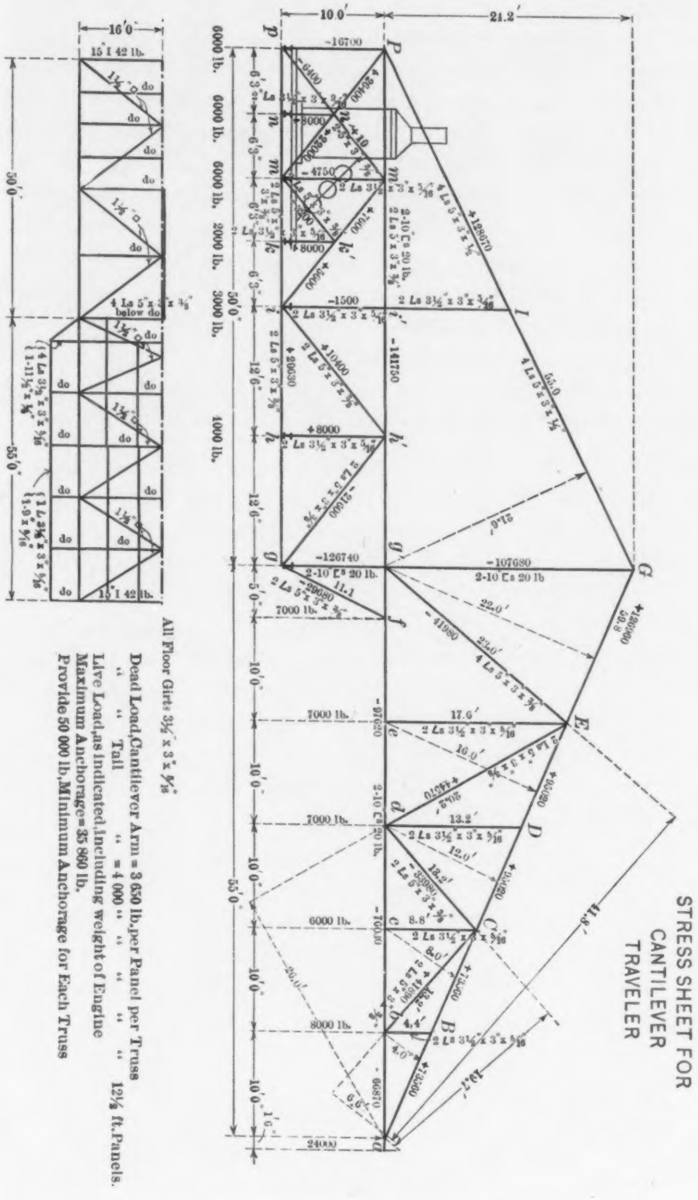

STRESS SHEET FOR
CANTILEVER
TRAVELER
HALF PLAN
All Floor Girts 3½" x 3" x 5/16"
Dead Load, Cantilever Arm = 3 650 lb. per Panel per Truss
" " Tail " = 4 000 " " " " " 12½ ft. Panels.
Live Load, as indicated, including weight of Engine
Maximum Anchorage = 35 860 lb.
Provide 50 000 lb. Minimum Anchorage for Each Truss

Fig. 8.

diagrams of the creeper traveler. It was carried over the toggle bars on I-beam stringers supported at U_{16}, U_{17}, and U_{18}. From U_{18} it was supported on a track laid on the upper chord of the suspended span. This traveler was so simple that further description seems unnecessary. It met the expectations fully, and all the members of the erection force were pleased with it.

Credit for the results secured from the design and construction of this bridge is due to all the engineers who were connected with the work, both on the part of the County and of the Bridge Company, and the writer received material assistance from them in the preparation of this paper. G. Gudmundsson, Assoc. M. Am. Soc. C. E., Consulting Engineer, was associated with Mr. J. G. Chalfant, County Engineer, in the preparation of the contract plans. H. R. Blickle, M. Am. Soc. C. E., is Secretary and Chief Engineer of the Fort Pitt Bridge Works.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

PORTS OF THE PACIFIC.

By H. M. Chittenden, M. Am. Soc. C. E.,
Assisted by A. O. Powell, M. Am. Soc. C. E.
To be Presented November 20th, 1912.

To describe any one of the larger ports of the Pacific Coast adequately would exceed the admissible limits of this paper. To describe them all with anything like minute detail is clearly impossible, and it is assumed that, in assigning this topic,* the intention was that it be treated only on broad general lines which should give a comprehensive view of the subject as a whole, rather than a study in detail. Even with this understanding, the scope of the subject is so vast that its compression into the compass of a professional paper seems little better than the preparation of a table of contents. The attempt has been made, however, and its results are presented herewith under the following headings:

I.—Strategic Relations of Coast Ports.
II.—Descriptive Data.
III.—Engineering Problems.
IV.—Administrative Systems.
V.—Plans for the Future.
VI.—Influence of the Panama Canal.

The paper will treat only of those ports north of the boundary between Mexico and the United States.

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

* This paper was prepared for presentation to the Annual Convention at Seattle, Wash.

I.—Strategic Relations of Coast Ports.

Commerce of the Pacific.—Since the days of Magellan, imaginative minds have pictured the Pacific Ocean as the future home of the world's commerce. There is something in the immensity of that ocean, in the present vastness of the population on one shore and the future vastness of that on the other, which conjures up visions of argosies such as the Mediterranean or the Atlantic has never known. The situation as it actually exists, however, makes this picture somewhat of an illusion. The very wideness of the intervening sea is a mighty barrier to economic intercourse across it. The breadth of the Pacific as compared with that of the Atlantic, depending on the latitude, is as two or three to one; and, while the cost of transit and loss of time may not be in the same proportion, there is still a wide disparity against the larger ocean. There are also the almost insuperable barriers of race and economic conditions on the Asiatic Coast. We do not admit the Oriental to our shores as we do the European. That immense source of traffic which has sustained its thousands of ships on the Atlantic is cut off on the Pacific. Likewise those Oriental countries have little to offer the traveler, which can compare with the lure of European civilization, ancient and modern. Tourist traffic across the Pacific is a bagatelle compared to that over the Atlantic. Finally, the deep poverty of the hordes who swarm the Asiatic shores, and the backward condition of industry there, are not promotive of vigorous commercial intercourse, for they offer relatively little to sell and less with which to buy.

Thus it results that the commerce of this ocean, great though it be, is small compared with that of the Atlantic, and small to what might be expected from the millions of inhabitants affected by it. Its growth in the near future, strange as it may appear, is more dependent on the countries which border the Atlantic than on those washed by its own waters. As far as the Pacific Coast is concerned, the eyes of its people are turned east rather than west. It is there that their kindred dwell; it is there that the capital exists which shall develop their boundless resources; it is thence that must come those who shall help to populate their shores. It is this fact—that their future is on the Atlantic rather than on the Pacific—which makes the opening of the Panama Canal an event of such tremendous import to them. It is bringing them to their own. It fronts them toward

Europe. It opens to them the treasures of the Occident—far more potent, if less romantic, than those of the Orient. It gives them something of the advantage which the eastern shore of the Continent enjoys by reason of its closer relation to the fundamental sources of our civilization.

If this condition is true of the present, and will remain so, to a degree, for the indefinite future, let it not blind us to the profound changes which are taking place in those ancient, and, but yesterday, non-progressive, countries which lie on the other side of the Pacific. When we consider what a volume of commerce the accidents of political fortune have turned in this direction from the distant Philippines; when we reflect on the marvelous progress of modern Japan; and when we note the amazing changes now going on in that venerable nation which has slumbered indifferent to the rest of the world for thirty centuries or more, we cannot afford to take a pessimistic view of our future relations with that side of the globe. If ever these people accomplish what is within their practical reach; if ever they turn to account, as is done in Europe and America, their wealth of natural resources and their own capacity for industrial development, surely there will result a mighty increase in trade between them and the rest of the world, and of this the American Continent will enjoy the greater share.

Thus, while the Atlantic holds out to us the brighter present prospect, there is brilliant promise in the Pacific, and in every direction the future is big with hope. Visions of the coming day are profoundly stirring the minds of men. Expansion is in the air. The measureless force of unseen psychological influences rushes the world along whether it will or no. Doubtless, it builds exaggerated hopes and paves the way to much disappointment, but its very exuberance of faith is an earnest of vast accomplishment.

This far-reaching movement, which has been crystallized into definite form by the approaching consummation of the greatest engineering work of ages, finds its intensest expression along the Pacific Coast of North America. To other parts of the world, the Panama Canal means simply increased opportunities for trade; to the Pacific Coast it means a new lease of life through the elimination of those barriers which separate it from its true source of sustenance and growth. Everywhere along the Coast, faith in the beneficial results

of this great work is unbounded. It is a faith, moreover, which expresses itself in works. From Alaska to Lower California, the Coast is getting ready for the Canal. It is putting its house in order. It is spending in this work prodigious sums of money. The present decade will witness an expenditure in port development of probably \$50 000 000. The slogan which has won this vast sum from the pockets of the taxpayers is: "Get ready for the opening of the Panama Canal," and the formal celebration of that event will find the work well toward completion.

It would be idle to pretend that this prodigious effort springs solely from an actual necessity of providing for the increase of traffic that will result from the opening of the Canal. The popular belief is, of course, that this is the case; but those who have studied the situation closely know better. They realize that the movement is being overdone, but they recognize that it is bound to keep on through the fear which each port entertains of what its sister ports may do. The stigma of possible failure in the race and fear of loss of prestige are the potent forces which are back of these extraordinary efforts. Los Angeles or Seattle would find it difficult on the cold basis of rational business foresight to justify their enormous prospective outlays; but they find ample justification in the necessity of keeping up with a procession which now stalks with tremendous strides from one end of the Coast to the other.

Competition of Ports.—This interesting state of things suggests a primary basis for comparing the ports of this Coast, namely, their relations to one another as commercial rivals. In the long stretch of coast line from Latitude 32° 30′ to Latitude 54° 40′, there are certain points of true strategic value; that is, points the possession of which gives the possessor important advantages over his competitor. Broadly speaking, there are two great divisions of the Coast from a commercial point of view—the north and south—with the Oregon-California line the approximate boundary between them. Only in a very general sense are these two sections of the coast competitors. They both reach out, it is true, to Hawaii, the Orient, and Alaska, and inland to common interior points; but each has some advantage peculiar to itself—a sort of proprietary right in certain spheres in which the other seems rather an interloper. The competition in such cases

is not keen, for the advantage of the more favored rival is too pronounced to be coped with successfully.

It is in a narrower sphere that the competition of these ports becomes really intense. San Francisco, Los Angeles, and San Diego have little to fear from the ports of the North, for their respective hinterlands merge only in the far interior of the Great Basin, but they have a wholesome respect for one another. Portland poaches but slightly on San Francisco's preserves, and Puget Sound still less, but the port on the Columbia never for an instant lets her sisters farther north forget her existence. Puget Sound and the ports of British Columbia are also competitors of a strenuous type in spite of the artificial barriers of nationality, tariff, and different customs and laws. In these local groups competition is really keen, and each competitor is feverishly anxious as to what its rivals are doing, while the still smaller units of the group flourish either by virtue of special advantages or simply on the crumbs which fall from their wealthier sisters' tables.

San Francisco Bay.—Having thus touched in most general terms on the trade relations of different sections of the Pacific Coast, let us consider in some detail the principal ports, still from the point of view of their commercial relations. Whatever changes the future may have in store, it is now true, and for a long while will so remain, that San Francisco Bay is far and away the most important port on the Coast. It is a wonderful port—wonderful in the strategic relation to its California hinterland and to the great interior of the country; wonderful in its physical conformation as a vast sheltered harbor opening in, through a narrow and easily defended entrance, from a coast line almost devoid of harbors for hundreds of miles in either direction; wonderful in its romantic history, and wonderful in its relation to the commerce of the world. Nature wrought a masterpiece when she made San Francisco Bay. Its great expanse and its navigable connections north and south, through the rich valleys of the San Joaquin and Sacramento, fit it perfectly as the entrepôt of a vast empire. The work of Nature was supplemented by the good offices of fortune, which early turned the attention of the world to this port and laid the foundation of its future greatness so deep that neither earthquake nor the growth of rivals can shake it. The Golden Gate—named three centuries before, in beautiful prophecy of

the Argonauts of Forty-nine, whose anchors dropped into yellow sands brought down by the slicken-laden streams of the Sierra—was the scene of a mighty commerce while yet only random traders sought the furry wealth of the harbors farther north. The first transcontinental railway had poured its traffic into the valley of the Sacramento for twenty years before any other portion of the Coast was similarly favored. San Francisco had written the most important chapter of her history while her sister ports were still almost unknown to the world. Congress did well when it selected the California metropolis as the site for the celebration of the opening of the great Canal. What a contrast it will be—the struggling mass of humanity and freight on its way across the fever-stricken Isthmus to the land of golden promise in Forty-nine, and the floating palaces which will then pass safely through Culebra Hill to a scene of resplendent riches undreamed of by even the wildest imagination of sixty-six years before! Where else on the round earth has modern progress wrought so great a change in so short a time?

Los Angeles and San Diego.—In Southern California the location of highest strategic value, from a commercial point of view, unfortunately, happened to be where Nature did not provide a harbor, while the one really good natural harbor on that section of the Coast is not located where it will best serve the commercial needs of the country. Los Angeles, by virtue of its relation to the great Southern trade routes east, its marvelous resources in agriculture, its close proximity to the oil fields of Southern California, and the inestimable wealth of its winter climate (not exclusive in this, however), is incontestably the center of activity of Southern California. So little did Nature do in the way of providing a port in that vicinity, however, that the town did not grow up on the seashore at all, but twenty miles inland, and the possibility of making it a genuine seaport was clearly an afterthought in its development. Now, this consummation is its chief ambition, and a superb effort is being made to realize it. Los Angeles will become a great port, not because Nature made it so, but because her own virile people have said so. Its future harbor will be almost wholly artificial, but it will be a great harbor nevertheless, and will stand all the more to the credit of its people because of the sacrifices which they will have made to obtain it.

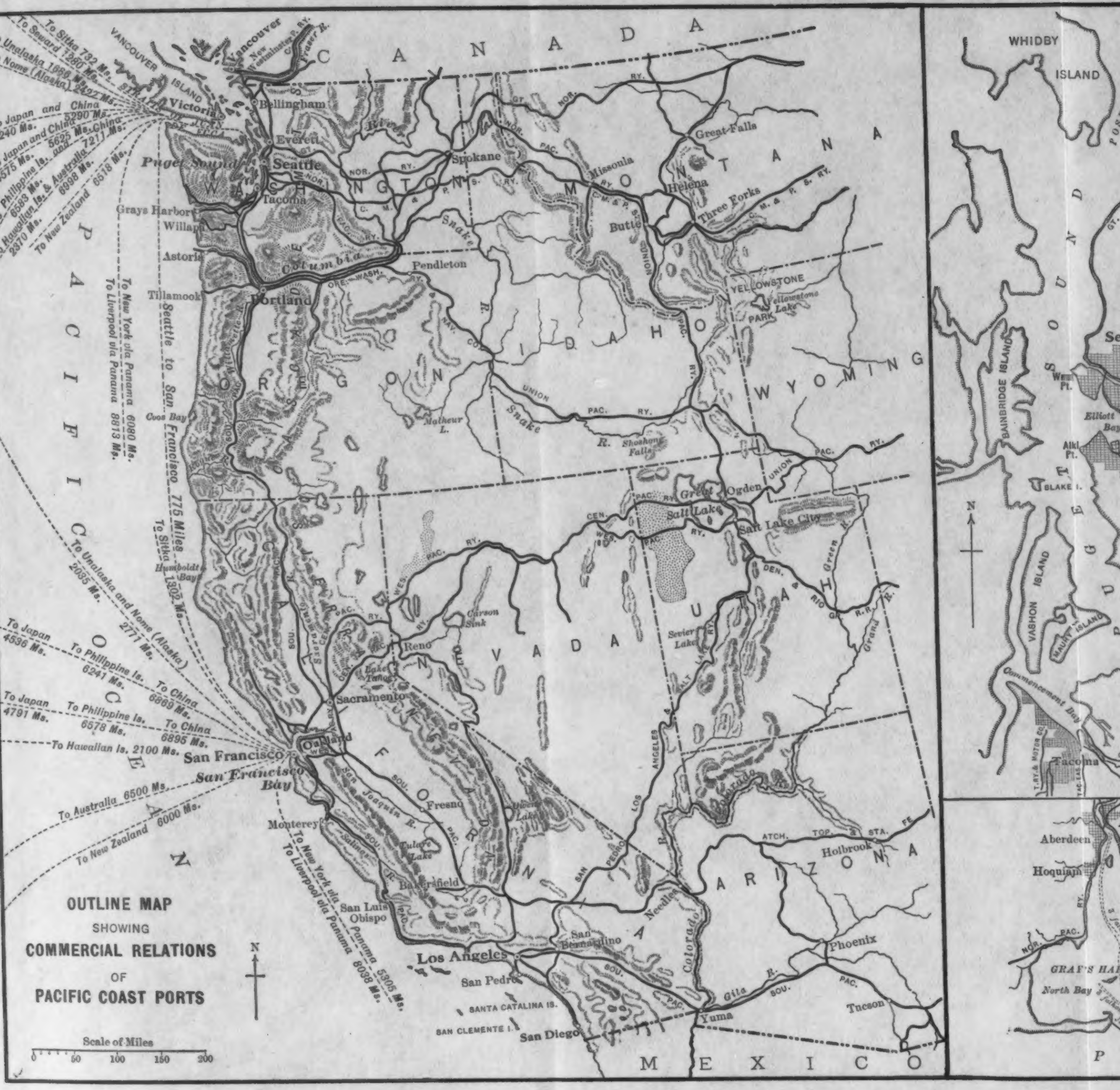
Southeasterly from Los Angeles, 89 miles, lies the splendid landlocked harbor of San Diego, ideal in its physical advantages. The fact that it never has become of first importance, however, shows that it requires something besides Nature's aid to make it a great port. It is not well located with reference to rail routes east; it has not as good a tributary country as Los Angeles; it is hemmed in by bold uplands close to the shore; and, worst of all, it suffers the purely artificial handicap of close proximity to the National frontier. Still, in spite of these drawbacks, the great natural advantages of the harbor, the lack of such advantages at Los Angeles, and the arbitrary rail tariffs which until recently have made freight from Port Los Angeles to the main city cost as much as from San Diego itself, though four times as distant, make San Diego a powerful competitor with her sister city to the north. Of course, she is bound to lose something of her relative advantage as Los Angeles develops her port, and particularly when the proposed municipal belt line connects that city with the sea and reduces freight rates to a nominal figure.

These two Southern ports have naturally cut somewhat into the trade of San Francisco, and will continue to do so. This is to be desired as far as they may better serve the needs of the public as a whole, and beyond that point they are certain not to prevail if the communities of San Francisco Bay do their duty. California is a State of vast extent and immeasurable resources, and its hinterland is all the country to the eastward. In this illimitable field and with the boundless ocean to the west, there will be sustenance for all, and the final balance among the ports will be adjusted on the basis of maximum efficiency of service to the public.

The Columbia.—Northward from San Francisco the first location of high strategic value is the Columbia River—the chief river of the Pacific Slope—which here breaks through the mountain barriers and opens a low-grade route to the interior of the country. It is the only point in United States territory where the great Coast-Sierra-Cascade Barrier is completely traversed by a water-grade route. The main valley extends directly back from the sea a distance of 75 miles, where its tributaries begin to spread out until they expand like an enormous fan, giving arterial highways to a water-shed of 250 000 sq. miles, with routes across the Continental Divide to the far-spreading country beyond. The main stream is susceptible of improvement for

navigation as far as into British territory, and its principal tributaries can be navigated for short distances; but the rapid fall of most of the streams, and the invariable accompaniment of rock rapids, make the liberal use of locks a necessity, and this presupposes heavy cost. In any event, it is certain that the river will be improved in the near future so that boats can ascend at all seasons as far as Central Washington, say, to Priest Rapids. This water route and the low-grade rail routes on either side, ramifying along the tributaries in all directions, give the Lower Columbia ideal communications with its hinterland, considering the exceedingly mountainous character of the country.

Plate XCV shows, to any trained eye, the immense advantage of the situation at the junction of the Columbia and Willamette Rivers. It is a great cross-roads. To this point vessels from the ocean, once over the bar, can safely ascend. Here, rail and river take up the route to the far interior. South is the rich Willamette Valley and Nature's land route to California. North is the Cowlitz Valley and the route to Puget Sound. The possibilities of the situation are enormous and such as will survive all competition. The chief drawback, as far as its commercial relations with the outside world are concerned, is its connection with the sea. The Columbia Bar, built up from the detritus of a vast and steep-sloped water-shed, was originally the most formidable and dangerous known to navigation, and the problem of opening and maintaining a safe and commodious channel across it is one of the most difficult in the whole range of river and harbor engineering. Only a moderate degree of success has been obtained so far, and that at heavy cost. Other plans have been suggested, such as cutting through the sand spit to an artificial harbor to be built south of the jetty, or through the isthmus on the north to the fine natural harbor of Willapa Bay; but they have never received serious attention, and the plan of deepening the bar by jetty construction will probably continue. With it must go, hand in hand, the project of deepening the river itself in its 100-mile channel from the bar to the port. The situation is something like that at Los Angeles in that Man must be relied on to make good the large deficiency of Nature. This, as the writer has elsewhere observed, the enterprising citizenship of Portland is resolved shall be done, cost what it may, to the end that there may be realized, even more than at


OUTLINE MAP
SHOWING
COMMERCIAL RELATIONS
OF
PACIFIC COAST PORTS
Scale of Miles
0 50 100 150 200
N
CANADA
MEXICO
PACIFIC OCEAN
WASHINGTON
OREGON
CALIFORNIA
IDAHO
MONTANA
WYOMING
NEVADA
UTAH
ARIZONA
To Sitka 732 Ms.
To Seward 1380 Ms.
To Unalaska 1686 Ms.
To Nome (Alaska) 2492 Ms.
To Japan and China 4240 Ms. 5290 Ms.
To Japan and China 4575 Ms. 5625 Ms.
To Philippine Is. and China 6583 Ms. 7211 Ms.
To Hawaiian Is. & Australia 2370 Ms. 8998 Ms.
To New Zealand 6578 Ms.
To New York via Panama 6080 Ms.
To Liverpool via Panama 8813 Ms.
Seattle to San Francisco 775 Miles
To Sitka 1302 Ms.
To Unalaska and Nome (Alaska) 2095 Ms. 2771 Ms.
To Japan 4536 Ms.
To Philippine Is. 6241 Ms.
To China 6869 Ms.
To Japan 4791 Ms.
To Philippine Is. 6578 Ms.
To China 6895 Ms.
To Hawaiian Is. 2100 Ms.
To Australia 6500 Ms.
To New Zealand 6000 Ms.
To New York via Panama 5305 Ms.
To Liverpool via Panama 8038 Ms.
Vancouver
Victoria
Bellingham
Everett
Seattle
Tacoma
Puget Sound
Grays Harbor
Willapa
Astoria
Tillamook
Portland
Coos Bay
Humboldt Bay
Spokane
Missoula
Great Falls
Helena
Three Forks
Butte
Pendleton
Yellowstone Park
Ogden
Great Salt Lake
Salt Lake City
Reno
Sacramento
Oakland
San Francisco
San Francisco Bay
Fresno
Monterey
San Luis Obispo
Bakersfield
Los Angeles
San Pedro
San Bernardino
Needles
Holbrook
Phoenix
Tucson
Yuma
San Diego
SANTA CATALINA IS.
SAN CLEMENTE I.
WHIDBY ISLAND
BAINBRIDGE ISLAND
VASHON ISLAND
MAURY ISLAND
Commencement Bay
Tacoma
Aberdeen
Hoquiam

POSSESSION BAY
Everett
Bothell
Seattle
Kirkland
Lake Union
Elliott Bay
LAKE WASHINGTON
Lake Sammamish
Renton
Swan L.
South Aberdeen
SAN PABLO BAY
Vallejo
MARE ISLAND
Benicia
SUISUN BAY
Carquinez Straits
Vallejo Junc.
Costa
Pinole Pt.
Pt. San Pedro
Pt. S. Pablo
San Quentin
Richmond
Mt. Tamalpias
Duxbury Pt.
Bluff Pt.
Sausalito
Lime Pt.
Pt. Bonita
Golden Gate
Fort Pt.
Pt. Lobos
San Francisco
BERKELEY HILLS
Berkeley
Oakland
Alameda
San Leandro
PACIFIC OCEAN
SAN FRANCISCO BAY
Hunters Pt.
Lake Merced
South San Francisco
Lake San Andrea
Pt. San Pedro
Pt. San Mateo
SCALE FOR PORT MAPS
Scale of Miles
0 1 2 4 6 8 10 12 14 16
National City
San Diego
SAN DIEGO BAY
North San Diego
Coronado
False Bay
Soledad Mt.
Pacific Beach
La Jolla Pt.
Pt. Medanos
U.S. MEXICO
Los Angeles
Santa Monica
Redondo
Wilmington
San Pedro Hill 1475
San Pedro
South Bend
WILLAPA BAY
Whaling Sta.
South Bay
COLUMBIA RIVER
ILWACO & SHOALWATER BAY RY.
3 fathom line

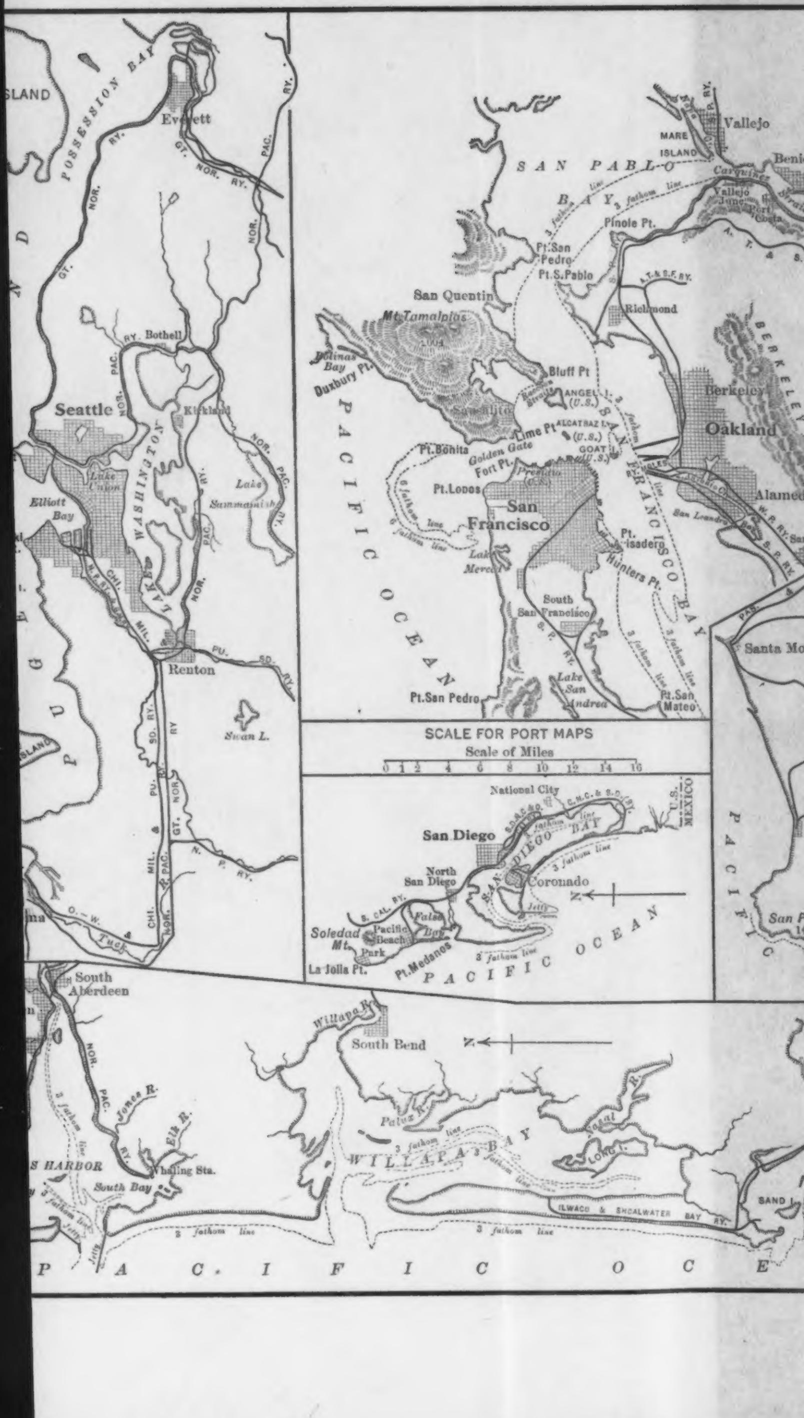
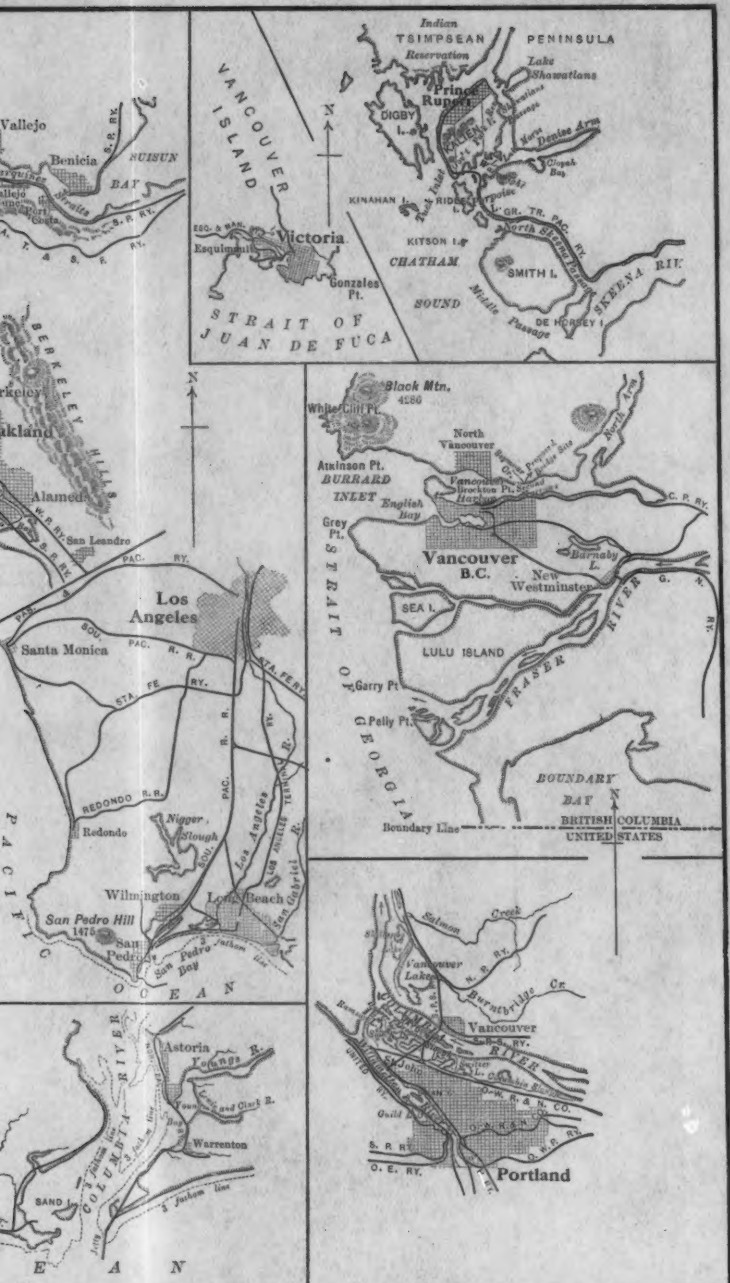

POSSESSION BAY
Everett
Bothell
Seattle
Kirkland
Lake Union
Elliott Bay
LAKE WASHINGTON
Lake Sammamish
Renton
Swan L.
PUGET SOUND
South Aberdeen
South Bend
Willapa R.
Jones R.
Elk R.
Whaling Sta.
South Bay
Palux R.
Nasal R.
WILLAPA BAY
LONG I.
ILWACO & SHOALWATER BAY RY.
3 fathom line
P A C I F I C O C E
SAN PABLO BAY
Vallejo
MARE ISLAND
Carquinez
Vallejo Junc.
Port Costa
Pinole Pt.
Pt. San Pedro
Pt. S. Pablo
San Quentin
Mt. Tamalpias
Richmond
Bolinas Bay
Duxbury Pt.
Bluff Pt
Berkeley
Oakland
Alameda
Pt. Bonita
Golden Gate
Fort Pt.
Lime Pt.
Pt. Lobos
San Francisco
Presidio (U.S.)
Lake Merced
SAN FRANCISCO BAY
Hunters Pt.
South San Francisco
Lake San Andrea
Pt. San Pedro
Pt. San Mateo
PACIFIC OCEAN
Santa Mo
SCALE FOR PORT MAPS
Scale of Miles
0 1 2 4 6 8 10 12 14 16
National City
San Diego
SAN DIEGO BAY
North San Diego
Coronado
U.S. MEXICO
Soledad Mt.
Pacific Beach
Park
False Bay
La Jolla Pt.
Pt. Medanos
PACIFIC OCEAN

PLATE XCV.
PAPERS, AM. SOC. C. E.
SEPTEMBER, 1912.
CHITTTENDEN ON
PORTS OF THE PACIFIC.



present, the prophecy of one of her own poets, who has pictured the time:

> "When through this Gate the treasures of the North
> Flow outward to the sea."

Puget Sound.—Proceeding northward, the next great strategic point in the commerce of the Pacific Coast, and, in some respects, the most important of all, is Puget Sound. It is a vast inlet from the sea through a strait 15 miles wide and nearly 100 miles long, branching both north and south, in sheltered inland waters which abound in ideal natural harbors. So perfect are these harbors that their chief defect is an excess of what anywhere else would be considered almost the chief advantage, the depth in some being such that anchorage ground is scarce, and extending so close in shore as to make wharf building difficult and expensive. The tidal range is also large with strong tidal currents, and the *teredo* is very active. Except for these drawbacks, the Sound harbors are ideal. Take Seattle, for instance. Into the spacious enclosure of Elliott Bay, a ship can enter without tug or pilot and pass directly to berth under her own steam in all conditions of tide, and in almost all conditions of weather, and always feel certain that her hull will not touch bottom. It is the most favored port in the world in this respect, excepting only one or two of its sister ports on the Sound, which enjoy the same advantage.

Nature has certainly lavished her richest favors on these ports and provided them with advantages which no expenditure of money can ever give to a port situated as is Portland or Los Angeles. But as an offset to this great advantage, she has placed a handicap which is of very serious consequence; she has barred off Puget Sound from its great hinterland to the east by a massive mountain range, the lowest passes of which are more than 3 000 ft. high. These passes have been scaled successfully, it is true, and the terminal rail rates are the same as to Portland, but the handicap is there nevertheless, and is the most serious commercial problem which Puget Sound faces to-day. It is as if the Catskill Range, with twice its actual height, extended along the coast in an unbroken wall from Boston to Baltimore, shutting out New York from its rightful hinterland. Local enthusiasts in both Portland and Seattle speak of their respective cities as the "New York of the Pacific." In truth, each is only half

New York; or rather, New York, in its natural advantages as a port, is the Seattle and Portland combined of the Atlantic. Its harbor rivals that of Seattle, while its Hudson River route to the interior rivals that of the Columbia. Puget Sound does not fully appreciate this difference. Seattle and Tacoma are spending millions in developing harbors which are already perfect beyond any in the world, but not a cent to overcome this barrier which stands between them and their hinterland. The writer once had the temerity to suggest that a low tunnel, serving all the railroads and giving practically a water-grade connection between the Sound and the Columbia Valley, was essential to the commercial supremacy of Puget Sound ports. Time alone will justify or discredit this prediction, but it is certain that with an outlay no greater than what has been expended on three of the Alpine tunnels, a line could be secured which would serve, on the most favored gradients and with shortest distances, the entire country from the Great Basin to the interior of Canada. Puget Sound is the natural entrepôt for the trade of Alaska, is a day nearer the ports of Japan and Northern China than San Francisco, and has the best harbors in the world. Only the natural barrier which separates it from its hinterland keeps it from playing the part which it should in the commerce of the Pacific.

We have considered Puget Sound as a great harbor whose entrance is the broad, deep Strait of Juan de Fuca. Inside the Strait there are numerous competing ports, of which Seattle is the central and most important. Thirty miles farther south is Tacoma, Seattle's most formidable competitor in United States waters. It has a harbor rivaling that of Seattle, and has been particularly favored by the great railway systems, which have given it an importance even beyond what its comparative advantages justify. Besides Tacoma there are the harbors of Everett, Anacortes, and Bellingham, all of which have splendid natural advantages and do a thriving trade. On the west side of the Sound is Port Townsend, once looked on as the coming port of Puget Sound, but now important mainly as the headquarters of the Customs District of the Sound and the Quarantine Station. It is on the wrong side ever to assume first importance.

All the great strategic points in United States territory, which we have considered, have been guarded by elaborate sea-coast defenses, while San Francisco and Puget Sound have been made great naval bases.

Vancouver, B. C.—Seattle's greatest rival for commercial supremacy north of Portland and Tacoma is Vancouver, B. C., a rival whose strength lies not so much in its natural advantages as in the artificial conditions arising from its being in a different national jurisdiction. It has a good strategic location, it is true, being at the outlet of the second greatest river of the Pacific Coast; but its harbor is inferior to those farther south, and its rail connections east, even to Central Canada, are inferior. Its early growth was due primarily to the fact that it was a Canadian port. That the port would have developed where, or as rapidly as, it has, if there had been no international boundary near by may well be doubted. As conditions actually exist, however, taken with the present status of American navigation laws by which cheaper foreign shipping is excluded from our coastwise trade, Vancouver is a formidable competitor with ports farther south. In the fish and lumber export trade, particularly, conditions tell heavily in her favor.

Prince Rupert.—Prince Rupert is the northernmost Pacific terminus of the transcontinental lines and will remain so until some line shall cross into the valley of the Yukon and descend that mighty river on its way to the westernmost apex of the Continent. It is 40 miles south of the international boundary of Alaska (54° 40′) and about 700 miles along the coast from Seattle and that much nearer Alaska. It is the westernmost, as well as the northernmost, transcontinental terminus, and is 500 miles (according to the statement of the Grand Trunk Pacific officials) nearer Asia than any other terminus on the Pacific. It is said that a traveler from China would be able to reach Winnipeg, *via* Prince Rupert, before he could reach Vancouver, if he were to go by that port. Add to this the fact that the gradient over the mountains, with the exception of about 20 miles of 1% on the west slope, is everywhere under five-tenths, and most of the way much less than this, and some of the physical advantages of the route are apparent. Add again the proximity of Prince Rupert to the limitless wheat fields of Canada, the fact that it is in the very center of the salmon and whale industry, among inestimable quantities of virgin timber, and its great future seems doubly assured.

Its advantages are offset to some extent by the severity of northern winters, but chiefly by the different nationality of the territory (Alaska) which it is best fitted to serve.

Secondary Ports.—Somewhat outside the rivalries of the great ports along the Coast, and flourishing on advantages which are peculiar to themselves, are several smaller ports. Among them, and the more prominent, are San Luis Obispo, about half way between San Francisco and Los Angeles; Humboldt Bay, on the North California Coast; Coos Bay, 200 miles south of the Columbia and serving an important section of Western Oregon; Astoria, just inside the Columbia Bar and the first port to be established on the North Pacific Coast; Grays Harbor, a great lumber port on the west coast of Washington, 45 miles north of the Columbia; and Victoria, an important harbor and naval base on Vancouver Island. There are numerous harbors on the Alaskan coast, but they are still in a state of Nature, as very little has been done toward converting them into up-to-date ports. The prospective opening of the coal fields of Alaska, and a more definite Alaskan policy on the part of the Government, will undoubtedly lead to the establishment of permanent facilities at some of these points in the near future.

II.—Descriptive Data.

In its physical characteristics the Pacific Coast line of North America increases in severity from south to north, but there is less diversity in climatic conditions than one might expect, for such great differences in latitude, owing to the moderating effect of the ocean currents. It is not until well up on the Alaskan Coast that one finds the harbors regularly sealed by ice in the winter season. Storms are more severe in the northern latitudes, but this is more than offset, as far as coastwise trade is concerned, by the sheltered inland passages which extend for at least 1 000 miles from Southern Puget Sound north. The tidal fluctuation increases from south to north from a mean of about 4 ft. at San Diego to 14 ft. at Prince Rupert. The *teredo* is very destructive in all the waters of the Coast, but the *limnoria* is active only in Californian waters. Nearly all the tributary streams are heavy silt carriers, and the primeval bays and inlets are partly or wholly filled up, making dredging a necessary adjunct of harbor development all along the Coast, and necessitating costly training dikes to scour channels across bars or shoals. In Southern California, the immediate shores were originally lightly timbered; but from Northern California north they were covered with magnificent forests. The coast line is re-

markably uniform and unbroken by indentations as far as the Strait of Juan de Fuca, but from there north the exact opposite is the case.

With these few observations on the broad physical characteristics of the Coast, more detailed consideration will be given to the several ports.

San Francisco Bay.—The Bay of San Francisco (including, in that term, San Pablo and Suisun Bays) has a total area of 420 sq. miles and a shore line of about 350 miles. The area exceeding 30 ft. in depth at low water is about 190 sq. miles. The extreme tidal range is 8 ft., and the mean is 4.3 ft. The tidal currents are strong in the Golden Gate and Carquinez Straits, amounting to 7 miles per hour at spring tides. The entrance to the Bay is about 1 mile wide and very deep, and is guarded by formidable defenses. In a half circle about the entrance on the ocean side, and some 6 miles distant, is a narrow bar with a ruling depth of 30 ft. at low tide and two crossings of over 40 ft.

The heavily silt-laden streams—the Sacramento and San Joaquin, with a water-shed of more than 60 000 sq. miles, extending 400 miles along the western slope of the Sierra—have brought down immense deposits into the Bay and are progressively reducing its area. It once extended far up the valley of each stream, but is now practically limited to the area below the Straits of Carquinez, for Suisun Bay is shoaled up so as to be of little use for navigation except on the through channel to the rivers above. The deposits have also invaded the Lower Bay and have shoaled the depth, over two-thirds of its area, to less than 18 ft. Due probably to the action of the inflowing tides, this shoaling has been forced mainly on to the east side of San Francisco Bay and the north side of San Pablo Bay, leaving the east shore, where Oakland, Berkeley, and Alameda now stand, quite unapproachable in their natural condition by any craft larger than a rowboat, while the shore of the San Francisco Peninsula has practicable depths for the largest shipping close in to the bank. It was mainly this condition that made the Port of San Francisco Bay develop where it did instead of on the east shore. Now that reclamation work is utilizing the shoal areas to make new lands by dredging out deep slips, shipping will gravitate more and more to the east shore in order to avoid ferry inconvenience.

The shoaling of San Francisco Bay is one of those great natural blessings which the unthinking are so accustomed to look on as a curse. One-tenth of its natural area, with deep connecting channels,

would serve every possible need of commerce, while the other nine-tenths would be of immeasurably greater benefit reclaimed and turned to industrial or agricultural use. Every cubic yard of earth washed down from the rugged slopes of the mountains is worth a thousand times more in those low areas, where it is turned to efficient use in the service of Man.

San Francisco Bay is perfectly sheltered from ocean storms, is not subject to flood effects except in the extreme upper portions, at the outlet of the Sacramento and San Joaquin Rivers, and is absolutely free from ice. There were originally several dangerous rocks in the channels, but these, for the most part, have been removed, the most important removal being that of Blossom Rock. In practically all respects the natural advantages of San Francisco Bay are of the highest order, and, if its human custodians are faithful to their trust, it is destined to remain for a long time the leading port on the Pacific Coast.

San Francisco proper and its port grew up on the narrow peninsula lying between the southern half of the Bay and the ocean, south of Golden Gate. As far as land communication to the north and east is concerned, the city is practically an island, and has to rely exclusively on ferry service. The water-front development extends from the north-east extremity of the peninsula around into the Bay, and as far south as the city and county boundary. The jurisdiction of the Board of Harbor Commissioners stops short at this point, which fact is clearly indicated on Plate XCV. At present the berthing space for all classes of vessels aggregates more than 10.5 miles. The several ferries to the east shore and other points land near the foot of Market Street, and carry 125 000 passengers daily. Exclusive of fairways or forbidden anchorage, there is, approximately, 100 sq. miles of available anchorage ground in the Bay.

Dock construction consists of three classes: Entire wooden construction, concrete subconstruction only, and complete concrete construction. Much trouble has been experienced with subaqueous concrete work, probably due to defective methods or inspection. Nearly all the sheds are built of wood, and their areas vary from 60 000 to 84 500 sq. ft. The State owns no dry docks, but there are two privately owned graving docks at Hunter's Point and two floating docks at the Union Iron Works Plant, besides the Naval Dock at Mare Island.

San Francisco is the only port on the Coast at which any great

amount of permanent sea-wall construction has been done. Work of this character has been carried on for the past twenty years. In that time, including work now in progress, nearly 13 000 lin. ft. have been built, at costs ranging from a little less than $100 to about $270 per lin. ft. The construction of the sea wall has resulted in reclaiming more than 25 acres, from which the annual rental now amounts to about $1 000 000, or considerably more than half the total revenue of the port.

The port's funds are derived from its regular revenues (rents, wharfage, dockage, tolls, etc.) and the sale of bonds. The regular revenues, from the beginning of the Board's operations in 1863, amount in round numbers to $28 500 000, and the funds from other sources, mainly bond sales, including recent issues still unexpended, to about $12 000 000. The expenditures for construction and repair amount approximately to $22 000 000, exclusive of the recent $10 000 000 bond issue. The running expenses of the port (salaries, law fees, etc.) amount to an average of 21.4% of all the disbursements. The total Federal appropriations for removing obstructions from the harbor amount to $516 000, so that the total outlay on the harbor, including recent bond issues, will be about $32 500 000. This, of course, is not all represented in actual present results, for much of it was for early work which has since been replaced, existing work probably not representing a present cost exceeding $10 000 000.

It is important to note that approximately one-half (of late years more than one-half) the revenue comes from ground rents, without which much larger bond issues would have been required.

The commerce of the port is very extensive. During the fiscal year 1908-09, 39 251 vessels of all descriptions were docked at the State wharves, and the short tons of freight passing over the wharves in the same period was 6 325 000. The Annual Report of the Engineer Department of the United States Army for the calendar year 1910 gives a total short-tonnage of 7 325 000, valued at $222 500 000, distributed as shown in Table 1.

The trade of San Francisco embraces every class of commerce that passes over the Pacific Ocean. The Government Transport Service on that ocean largely centers there, and the chief naval base of the Pacific Coast is at Mare Island in San Pablo Bay. In many other respects San Francisco stands far in the lead of any other port on the Coast.

TABLE 1.—FREIGHT TRAFFIC.

Articles.	Amount, in short tons.	Valuation.
Flour	98 089	$6 670 052
Corn	6 320	221 200
Wheat	143 104	5 724 160
Barley	543 407	16 302 210
Oats	29 869	896 070
Beans	46 913	3 753 040
Hay	214 734	4 079 946
Onions	16 815	1 325 200
Bran	16 865	573 410
Potatoes	98 415	3 936 600
Lumber	1 599 275	20 790 575
Shingles	34 037	1 157 258
Brick, cement, lime, etc	62 814	1 307 038
Coal, coke, etc	623 174	7 276 328
Crude oil	2 786 980	13 934 900
Paper	22 107	2 210 700
Wines, liquors, etc	101 269	20 253 800
Fertilizer	25 170	830 610
Salmon, codfish, etc	89 180	15 723 656
Butter, eggs, cheese, etc	26 984	11 436 440
Fruits, dried fruits, etc	86 553	10 470 950
Sugar, tea, coffee, canned goods	295 089	38 342 350
Silk, cotton, dry goods, leather	31 159	13 370 800
Wool, gunnies, matting, jute, etc	29 182	5 836 400
Rice, salt, coal oil, etc	33 709	1 849 205
Railroad ties, pig iron, etc	116 302	1 528 762
Miscellaneous	147 062	12 676 488
Total	7 324 577	$222 478 148

Oakland and Alameda.—The physical characteristics of San Francisco Bay, of which Oakland and Alameda occupy the east shore, have already been described. The distinctive east shore harbor is the San Antonio Estuary, which lies between Oakland and Alameda and extends entirely around the land side of the island on which Alameda is situated. It is referred to in Government reports as Oakland's inner harbor. A channel leads in, from deep water in the Bay, between jetties 800 ft. apart, which also serve as moles with ferry slips at the ends. The channel, as developed under the Government project now in force, will be 500 ft. wide and 30 ft. deep to a large "tidal basin" about opposite the center of Alameda, with a channel 300 ft. wide and 25 ft. deep entirely around the basin, and a depth of 18 ft. in the "tidal canal" connecting the basin with San Leandro Bay, an inlet of San Francisco Bay on the southeast end of Alameda Island. This magnificent inner harbor has a shore line on each side of about 7 miles. On the Oakland side there is approximately 10 000 ft. of berthing space for deep-sea vessels and 16 000 ft. for smaller craft. The harbor is a great industrial center, some of the largest plants on the Bay being located there. There are four dry docks.

From the point of junction of the Estuary with the Bay, the shore line to the north is occupied by transcontinental railroad terminals and shops of ferry lines. In this stretch of about 2½ miles frontage is the so-called western water-front on which the Municipal Government is making improvements.

The port's revenues come from tax levies, bond sales, rentals, fees, etc. The City Government has recently issued \$2 500 000 in bonds, and contemplates duplicating this issue in the near future.

The commerce of the port is large and varied, amounting, in 1910, to 3 575 000 short tons, valued at about \$140 000 000, of which approximately one-half was carried in merchant vessels and one-half in ferries.

All along the east shore, and at many other points on both shores of the Bay and of San Pablo and Suisun Bays, there are wharves and industrial establishments of one kind or another, of which only mention can be made here.

Los Angeles.—As already observed, Los Angeles was less favored by Nature as a port than any other important locality on the Coast, and the making of a great port there is the work of its enlightened citizenship. The harbor was an open bay protected only from westerly and northerly winds. It lay wide open to an exposure of 120° toward the south except for the meager influence of Catalina Island, 18 miles in the offing. From this bay, through a long crescent-shaped bar, formerly called Rattlesnake Island, now Terminal Island, there opened inland by a channel only 3 ft. deep at low tide what has come to be under development the inner harbor of Los Angeles. It is really the delta of Los Angeles and San Gabriel Rivers (erratic streams characteristic of the water-courses of that section of the country) which bring down heavy loads of silt in their periods of sudden floods. The outer harbor, being exposed to storms from the southwest, required artificial protection, and this has been provided by one of the largest breakwaters in the United States, extending from Point Fermin, previously referred to, easterly, a distance of 11 000 ft. to a low-tide depth of 48 ft. The space thus enclosed, measured back from a line drawn from the end of the breakwater perpendicularly to the shore, is 960 acres, and of this more than 400 acres has a depth of over 30 ft. The anchorage is good, the kelp which formerly was a great annoyance has died out, and the harbor thus formed serves its purpose excellently well. The mean tidal range is 4.1 ft., and the extreme 10 ft.

Great reliance for port facilities, however, is being placed on the inner harbor, which is almost wholly an artificial creation. Its entrance from the outer harbor is completely sheltered by the breakwater. Beginning with 700 ft., it widens to 1 000 ft. at a point 3 000 ft. in, and then rapidly narrows to 500 ft., which width it retains to a point 7 500 ft. in from the outer end. The width then increases somewhat, and the channel terminates at a distance of 12 000 ft. from the entrance in a turning basin 1 600 ft. in diameter. From the turning basin channels lead off to the main interior docks known as the East and West Basins, respectively, which, with their future ramifications, will become the center of Los Angeles Harbor. The situation can be better grasped by reference to Plate XCV. The present width and depth are 400 ft. and 20 ft. to "the foot of the wharves"; thence 25.5 ft. to the turning basin. The existing Government project contemplates a depth of 30 ft. up to and including the turning basin, and two channels, 200 ft. wide and 20 ft. deep, from the turning basin to the East and West Basins.

As already stated, the harbor is almost wholly of artificial creation, and it is costing vast sums, entirely apart from docks and wharves, to arrive at a point which Nature unaided has given to most of the other ports on the Coast. Los Angeles, though liberally aided by the Federal Government, is helping herself effectively. It is a fine example of a virile civic spirit refusing to be balked by Nature in the matter of access to the sea. It will eventually rank with such ports as Hamburg and Glasgow in the extent to which Man by his energy and persistence has overcome the deficiencies of Nature.

The total development of berthing space is about 19 000 ft., most of which is directly controlled by the railroads. There is no dry dock. The wharves are principally of timber construction, but in recent years there has been a tendency to extend the use of concrete and steel, and the recent controversy between the Board of Public Works and the Board of Harbor Commissioners turned largely on the choice between concrete and creosote piling for the substructures. There is no sea-wall construction, unless the great breakwater be classed as such.

The local funds for port development, so far, have come entirely from bond issues, but ground rent will doubtless become a source of revenue in the near future, together with the revenues from docks which the port authorities are planning to build. Local expenditure to

date, including that of the Village of Wilmington prior to annexation to Los Angeles, and including also outstanding contracts, amounts to $625 000. The City has voted $3 000 000 in bonds, and it gave a pledge to the Cities of Wilmington and San Pedro prior to annexation that it would expend not less than $10 000 000 in port development. The expenditure by the United States on the outer and inner harbors, including outside authorizations, is:

Outer harbor	$3 078 000
Inner harbor	2 308 000
Total	$5 386 000

A great saving fact in the enormous labor of building the Harbor of Los Angeles is its close relation to the industrial development of the city. The lands into which the waterways are being dredged are admirably adapted for factory locations, and the material of excavation is being used in making the necessary fills. The whole development goes hand in hand in a way to produce the best results.

The trade of Los Angeles is active and is steadily increasing. The principal articles of commerce are oil, cement, and lumber products. The estimated short-tonnage and valuation for the calendar year 1910 are given in Table 2, the totals being 1 709 000 tons valued at $47 000 000.

As yet no public terminal or belt-railway system has been constructed in Los Angeles, but the city is taking steps to acquire one which shall serve the port and connect it with the city.

San Diego.—Nature has provided for San Diego, in the first instance, what Los Angeles with the expenditure of many millions can scarcely expect ever to equal. In general exterior outline the two ports are quite similar. Coronado Beach corresponds to Terminal (old Rattlesnake) Island, Point Loma to Point Fermin, while the entrances to the inner harbors are similarly situated in both cases. The difference is that at San Diego the entrance had a very good depth and ample shelter to begin with, while the inner harbor was already dredged. At Los Angeles there was a depth of 3 ft. at the entrance and no harbor to speak of. If San Diego had been as favorably located with respect to its hinterland as Los Angeles, the natural advantages of her harbor would have been decisive, always provided a virile and wide-awake people were charged with its development.

TABLE 2.—FREIGHT TRAFFIC.

Articles.	Amount (customary units).	Amount, in short tons.	Estimated valuation.
Lumber	579 842 876 ft. b.m.	1 014 724	$12 756 543
Shingles	243 595 000 pieces	48 719	487 190
Shakes	4 204 000 pieces	2 803	60 958
Lath	70 460 000 pieces	19 572	147 966
Ties	696 308 pieces	44 563	445 640
Piles	18 317 pieces	17 584	366 340
Poles	43 526 pieces	36 561	652 890
Posts	79 565 pieces	2 387	59 673
Logs	7 940 pieces	5 558	55 580
Doors	10 578	529	21 156
Staves		525	15 750
Lumber trucks	182	75	4 550
Grain		30 700	921 000
Flour		3 523	176 150
Sugar		2 028	202 800
Paper		4 600	391 000
Asphalt		9 587	67 109
Coal and coke		6 976	71 390
Cement	225 350 bbl.	45 070	450 070
Pig iron		3 454	51 810
Oils	1 301 302 bbl.	259 467	2 138 571
Wool	311 bales	77	23 100
Plaster	79 600 bags	3 748	37 480
Salt	741 bags	78	2 028
Rice	6 170 bags	308	20 533
Brick	377 000 pieces	942	3 328
Chemicals, drugs, etc		488	97 600
Manufactured iron and steel		5 964	1 192 800
Fruits and nuts		1 399	279 800
Canned goods	9 560 cases	239	26 680
Fertilizers		5 010	143 886
Hay and feed		159	2 385
Scrap metals		1 702	34 040
Wines and liquors		424	84 800
Live stock	11 211 head	3 080	213 792
General merchandise		126 671	25 334 200
Total		1 709 294	$47 040 588

The area of San Diego Bay inside the entrance is 24 sq. miles, of which 1 600 acres is more than 30 ft. deep; and the length of shore line is 38 miles. The original depth of 21 ft. in the entrance has been increased by the Government to 30 ft., and the present project contemplates an enlarged channel of this depth 600 ft. wide. The present berthing space amounts to 4 500 lin. ft., with a low-water depth of 22 ft. All the piers are private property, but they are subject to sequestration by the City at any time on payment of actual value. There is no dry dock and no public terminal railway. There is a marine railway for vessels up to 1 000 tons burden. The Government maintains a torpedo and submarine boat station in the harbor, together with quarantine and coaling stations. There is an important ferry service between the city and Coronado Beach.

The present sources of revenue to the port are bond sales, of which only one (for $1 000 000, voted November, 1911) has ever been au-

thorized. The total Government appropriation for the improvement of the harbor is $810 000.

The commercial business of the port is shown in Table 3, which embraces the calendar year, 1910. The totals are 400 000 tons, valued at $22 500 000.

TABLE 3.—FREIGHT TRAFFIC.

Articles.	Amount (customary units).	Amount, in short tons.	Estimated valuation.
Lumber	80 066 000 ft. b. m.	140 115	$1 761 452
Crude oil	466 888 bbl.	88 419	350 166
Cement	15 234 bbl.	3 047	30 468
Wool	34 000 lb.	17	6 800
Salt	....	392	10 192
Ores	....	2 714	542 800
Manufactured iron and steel	....	1 811	362 200
Brick	618 160 pieces	1 545	4 950
Grain	....	445	13 350
Fertilizer	....	1 634	42 484
Coal and coke	....	38 954	154 057
Live stock	11 043 head	3 036	147 984
Pig iron	....	2 961	44 085
Onyx	....	16 510	119 430
Sulphate of potash	....	1 774	66 269
General merchandise	....	94 674	18 934 800
Total	....	398 048	$22 591 487

Secondary Harbors South of the Columbia River.—Between Los Angeles and San Francisco is the important secondary harbor of San Luis Obispo; and between San Francisco and the mouth of the Columbia are the important secondary harbors of Humboldt Bay, California, and Coos Bay, Ore. These harbors, roughly speaking, divide by equal intervals, the long stretch of coast line between the principal ports. At San Luis Obispo the development work by the Government is in the nature of a breakwater to afford shelter; in the two Northern harbors, it consists of jetty construction designed to deepen the natural channels over the bars. In all three ports the outlay has been heavy in proportion to the commerce. Some of the most successful jetty work on the Coast is at Humboldt and Coos Bays, but it is impossible, in this paper, to make more than a passing reference to it.

The Port of Portland.—This is the only port of those considered in this paper which, in its physical characteristics, is strictly a river port. It is so far inland that tidal fluctuation, though perceptible at periods of low water, is negligible. On the other hand, it is subject to all the fluctuations of level characteristic of rivers above tidal influence, and the range between extreme high and extreme low water

is more than 30 ft. The harbor proper is not on the Columbia, but on the Willamette, about ten miles above its mouth, though, in general terms, looking to the future, the whole of both streams, from Vancouver, Wash., to Portland, may be regarded as the port of the Columbia Valley. Being a river harbor, the whole trend of its development shapes itself to that fact, and its counterpart is to be found in such a port as New Orleans rather than in those which have been considered in this paper. Quay development, parallel to the current, is the rule instead of slips or piers projecting into the stream. While diurnal fluctuations are absent, and no provision therefor is necessary in dock construction, the great seasonal fluctuations introduce even more serious difficulties in providing for differences of level.

The length of available berthing space for deep-sea shipping is about 6 600 ft. on the east side and 8 500 ft. on the west side. The corresponding figures for river boats and the mosquito fleet are 4 725 and 6 040 ft., respectively. Railroads own 25 800 ft. of water-front on the west side and 17 150 ft. on the east side. The port has two floating dry docks, but there is no municipal or belt railway.

As is often the case with a port situated far inland on a river, the problem of providing an adequate channel to the sea is the most troublesome. With 110 miles of river channel in which a low-water depth of 10 ft. was all that could be depended on originally, and with a wild and tempestuous bar at the river mouth over which the natural depth was only 20 ft., the problem of securing and maintaining a channel was the all-important one. The work on the bar must take rank among the greatest, if not the greatest, of the kind ever constructed. It has all been done by the Government. The channel improvement from the bar to Portland has been done jointly by the Government and the port in proportions of expenditure as shown below. There is at present a depth of 26 ft. at low water, and the Government is about to commence work to increase this to 30 ft. The Government now has three dredges available for the work, and the Port of Portland has two, and it is expected that this fleet will be soon increased by two more.

The port's revenues come from tax levies, bond sales, dry-dock charges, dredging, and a few minor sources. Nothing is derived from dockage, wharfage, and similar charges. The total amount of funds provided by the Government and locally is approximately as follows:

By the Government—The Columbia Bar......	$10 145 000	
" " " River Channel to Portland	2 725 000	$12 870 000
By the Port of Portland, approximately.................		3 700 000
By the Portland Dock Commission (bonds voted, but not yet expended)		2 500 000
Total		$19 070 000

As already stated, Portland is admirably situated on the land side to develop a large port business, but the channel difficulties between the port and the sea somewhat offset this advantage. The commerce, nevertheless, is large and varied, as will be seen from Table 4, the totals being:

Exports and imports..	415 522	short tons valued at	$8 451 414
Domestic	2 025 434	" " " "	31 313 990
Local	5 393 317	" " " "	25 116 602
Totals...........	7 834 273		$64 882 006

Grays Harbor.—The most important of the secondary harbors of the Coast is Grays Harbor, forty-five miles north of the Columbia. It is a pear-shaped body of water nearly 100 sq. miles in area. It is really a vast mud flat, much of it being bare at low tide, and the channel to the upper portion where the Towns of Hoquiam, Aberdeen, and Montesano are located, has to be maintained by dredging. Extensive jetty work has been built and is now in progress to develop a channel over the bar. The port has three railroads. Its importance depends primarily on its lumber trade, and it is the largest cargo lumber port in the world. The Government has appropriated about $2 400 000 for improvement of the harbor and bar. The commerce for 1910 amounted to 670 000 tons valued at $6 000 000, of which more than 80% represented lumber products.

Puget Sound.—In their physical characteristics the Puget Sound ports are much alike, the water being very deep, anchorage ground scarce owing to depth, the tidal range large, and the *teredo* active. As a rule there is absolute freedom from obstructions in the channels of approach. Pilots and tugboats (except the latter for sail boats)

are unnecessary, though pilotage on the larger vessels is customary. In all cases the streams leading into the harbors from the Cascades bear heavy loads of silt and have filled up the estuaries for long distances. The tide flats thus built up have proven an invaluable asset, for they have made up to some extent the deficiency of low flat lands necessary for industries and terminal facilities. South of Deception Pass rock is scarce, but north of that point it outcrops everywhere, and in some instances forms dangerous obstructions. The whole

TABLE 4.—FREIGHT TRAFFIC.

Articles.	Amount (customary units).	Amount, in short tons.	Valuation.	Rate per ton-mile.
EXPORTS AND IMPORTS.				Mills.
Bags and burlap	5 230 tons	5 230	$488 934	0.694
Cement	80 852 bbl.	19 202	119 061	0.122
Coal	18 733 tons	18 733	47 313	0.694
Coke	4 021 tons	4 021	14 187	0.308
Fibers	1 703 tons	1 703	190 313	0.793
Flour	283 679 bbl.	27 800	1 186 654	0.638
Fire brick and clay	7 149 tons	7 149	33 848	0.191
Iron, pig and bar	8 774 tons	8 774	137 454	0.184
Lumber	122 711 000 ft. b. m.	159 524	1 505 497	0.620
Machinery	50 tons	50	1 445	0.694
Rice	2 082 tons	2 082	90 662	0.851
Sulphur	7 630 tons	7 630	124 643	0.638
Wheat	5 120 826 bushels	153 624	4 511 403	0.421
Total		415 522	$8 451 414	
DOMESTIC.				
Lumber	317 932 977 ft. b. m.	413 313	$4 183 130	3.331
Logs	48 000 000 ft. b. m.	96 000	288 000	1.50
Oil	6 900 000 bbl.	1 159 200	5 477 600	2.50
Miscellaneous	356 921 tons	356 921	21 415 260	5.00
Total		2 025 434	$31 313 990	
LOCAL.				
Coal	3 424 tons	3 424	$23 968	20.00
Fish	13 223 tons	13 223	1 586 760	20.00
Flour	107 397 bbl.	10 525	526 250	20.00
Fruit	3 478 tons	3 478	208 680	40.00
Grain	728 413 bushels	18 116	391 525	25.00
Hay and feed	11 483 tons	11 483	172 245	20.00
Hops	135 tons	135	27 000	50.00
Live stock	22 001 heads	4 592	918 400	35.00
Logs (towed)	1 035 295 704 ft. b. m.	2 070 591	6 211 773	5.00
Lumber	13 930 960 ft. b. m.	18 110	181 100	20.00
Manufactured iron and steel	7 369 tons	7 369	257 915	25.00
Merchandise	55 146 tons	55 146	5 514 600	50.00
Miscellaneous	70 247 tons	70 247	4 214 820	50.00
Oil (barged)	1 872 582 bbl.	314 594	1 343 782	30.00
Piles (towed)	14 017 887 lin. ft.	280 358	1 121 482	12.50
Potatoes	30 610 000 lb.	15 305	306 100	30.00
Sand and gravel (barged)	1 059 133 cu. yd.	1 592 671	796 335	8.00
Shingles	2 198 733 bundles	38 390	1 151 700	28.00
Stone (barged)	562 364 cu. yd.	843 550	84 355	10.00
Wood (barged)	14 603 cords	21 905	54 762	15.00
Wool	105 tons	105	23 100	50.00
Total		5 393 317	$25 116 602	

conformation of the Puget Sound country is the result of glacial action, and many of the lakes and bays are simply the gouged out paths of glaciers, while the shores are masses of the most heterogenous material. So irregular and broken with indentations and islands is the Sound area that tidal currents and stages are wholly irregular. Tidal predictions are possible only as a result of long observation, and are stated in the Government tables in terms of retardation behind the Port Townsend prediction.

With this general reference to the ports of Puget Sound, a somewhat detailed description will be given of its principal port, Seattle, Wash. Seattle Harbor means primarily Elliott Bay, a not very large indentation into the east shore of the Sound. Between its extreme limits, West Point and Alkali Point, the pier-head line is 10 miles long. The really practical limits of the harbor, as far as the near future is concerned, may better be taken as Duwamish Head and the southern point of Magnolia Bluff, directly opposite. Within the line joining these points is Elliott Bay proper, quite similar in contour (though slightly smaller in area) to Commencement Bay, on which the port of Tacoma is situated. The area of Elliott Bay, as thus limited, is 4.3 sq. miles and its shore line (pier-head line), not counting slips and waterways, is 6.5 miles. Over this entire area there is ample depth for the heaviest shipping. There is never any ice nor rough weather to disturb deep-sea vessels, though southwest winds disturb small vessels in the reach south of Smith's Cove, as north winds do those on the south side of the harbor. The tidal currents are too slight to be drawbacks of any moment.

The water-front development is most pronounced along that portion where the city began its growth. For upward of a mile the entire frontage is in active use. There are some important piers, which, as a rule, are run out at very oblique angles with the shore in order to secure the necessary length without getting out to prohibitory depths. At the north is the important pier of the Great Northern Railway, and near it a still unimproved waterway, Smith's Cove, 1 mile long and 400 ft. wide, which is destined to important use in the growth of that part of the city. On the south side of the harbor, the delta of the Duwamish River has been filled in except for the reservation of two waterways, the East and the West, inclosing a tract of land popularly known as Harbor Island, from which great things are expected in

the future growth of the city. As the plan will finally work out, East Waterway will be practically a slip about 1 mile long and 750 ft. wide, with slips and quays on the two sides. The east side of the Waterway is really a part of the main water-front. West Waterway will be 900 ft. wide and will be the outlet of the Duwamish River.

The valley of the Duwamish furnishes the single large tract of bottom land directly adjacent to the harbor, and it must naturally develop into an important industrial center. Plans are perfected and well under way, with a large part of the funds already provided, for straightening the river and excavating a channel 4 miles up the valley to a bottom width of 150 ft. and a low-water depth of 16 ft. The utility of this work in the industrial development of the city is bound to be great.

Such is what may be termed Seattle's outer, or tidal, harbor, with a perimeter, including the three main waterways, of 12.5 miles east of Magnolia Bluff and Duwamish Head, and this is susceptible of almost indefinite extension by pier and slip construction. Every foot of this frontage is served, or readily capable of being served, by railroads, and the possibilities of expansion are all that are likely to be required for a century to come. The extent of existing wharf construction is such as to give about 14 500 lin. ft. of berthing space.

In addition to this magnificent gift of Nature, Seattle has another absolutely unique among the sea ports of the world—what may be called her inner harbor, soon to be opened to the commerce of the seas. The glaciers, in carving out the picturesque topography of this city, left two remarkable troughs or basins, bordered by hills from 200 to 500 ft. high. These basins are just enough above sea level to keep out the tides, and their waters, therefore, are fresh. Their own levels differ by only about 10 ft., the larger lake being the higher. Nearly all the area of both lakes is deep enough for the heaviest shipping; and all that Nature has left undone to make it accessible from the sea is to provide a lock and an artificial channel from Lake Union to the Sound, and a channel between the lakes. This work is being done by the Government and the local community. A channel, 100 ft. wide on the bottom and 36 ft. deep, will be provided with two parallel locks, one 825 ft. long by 80 ft. wide, interior dimensions, and the other 150 ft. long by 30 ft. wide, for small boats. Salmon Bay, a tidal slough between Lake Union and the Sound, will be raised by the lock

and dam to the level of Lake Union, and Lake Washington will be lowered to the same level, thus giving a uniform level from the lock to the remotest corners of Lake Washington. The canal right of way will admit of double the present proposed width, should commerce ever demand it. The bridges will have 30-ft. clearance, except one railroad bridge which will remain open when not in use. The fluctuation of level will not exceed 2 ft. The lakes are free of obstructions and do not freeze. Incidentally, the lowering of Lake Washington will absorb the floods of its water-shed and of Cedar River, and thus furnish a needed relief to the floods of the Duwamish, besides draining extensive marsh areas around the shore and making them available for industrial use.

There has been much speculation as to the value of this proposed improvement work. On the face of it, it would seem to furnish one of the most ideal opportunities that any port ever had. Think what it would mean to Los Angeles, if, instead of spending tens of millions for a narrow and restricted tidal harbor, she could, at an expense of $4 000 000 or $5 000 000, open up 40 sq. miles of fresh, deep water with a shore line of 100 miles. But at Seattle it does not mean what it would at Los Angeles. The rare perfection of the harbor of Elliott Bay places even this wonderful fresh-water harbor at a discount, as far as over-seas shipping is concerned. Still, its incomparable advantages are bound to tell. Absolute freedom from tidal fluctuation and currents, freedom from the *teredo* and, therefore, assurance of permanence in subaqueous timber construction, the beneficial effect of fresh water on the hulls of vessels, and the access which the lakes give to all sections of this district, are advantages which will not go unused. The first and immediate effect will be on small-boat traffic, lighterage, and the like, and a complete readjustment of this class of traffic may surely be expected. Salmon Bay and Lake Union will become great docks in the very heart of the city, while Lake Washington will offer thousands of acres of shore lands adapted to manufacturing. Gradually, larger shipping will follow, particularly in quest of lumber cargoes. Already inquiries are being made for sites for the fishermen's fleet and the yachting and motor-boat fleets. The large number of vessels which ply these northern waters, but which lie idle in winter, will find ideal refuge here. It will be strange if even the Navy Department does not take advantage of these fresh,

sheltered waters to lay up such of its vessels on this Coast as remain for a time out of commission. The ship-building industry of the entire Northwest ought to center at Seattle. Therefore, in spite of the unrivaled attractions of Elliott Bay, Seattle's future fresh-water harbor seems bound to become a priceless adjunct to the city's growth.

The natural perfection of Seattle's outer harbor is in no way better shown than by the remarkable fact—probably unique among American harbors—that no Federal appropriation has ever been made for its improvement. The only expenditure, not by direct private initiative, has been the development of the East and West Waterways and Harbor Island, and this has cost, in round numbers, \$3 000 000.

The development of the inner harbor will have cost, when completed, exclusive of bridges and other crossings, approximately \$4 250 000, of which the Government will have contributed about \$2 750 000, and local agencies (State and County) about \$1 500 000.

On the west shore of the Sound, directly opposite Seattle, and concealed in a deep inlet, with narrow, tortuous approaches, yet safe for the heaviest ships and protected by strong defenses, is one of the most interesting spots in the city. It is the Naval Station of the North Pacific Coast, destined to expansion, because of its superior advantages, into one of the greatest, if not the greatest, in the United States. It is mentioned in this connection with particular reference to its dry-dock facilities, which are often utilized by commercial vessels. There are other dry docks on the Sound, particularly the Heffernan Dock on East Waterway, Seattle, but no others of a capacity for docking the largest vessels.

Tacoma, the second port in importance on the Sound, is 30 miles south of Seattle, on Commencement Bay, similar in form and position to Elliott Bay. The tide flats have been built up by deposits from the Puyallup River, as those of Elliott Bay were by the Duwamish. Two waterways have been laid out, and one of them, the City Waterway, has been excavated into these flats. An attempt was made to excavate the other—the Puyallup Waterway—but the excessive deposits of silt brought down by the river (greatly augmented since 1906 by the bodily diversion to the Puyallup, through natural causes, of the White River from its former outlet into the Duwamish) made it impossible to secure or maintain the desired depth. The principal dockage facilities consist of a long quay dock on the west

side of the Bay, the City Waterway, and a slip excavated by the Chicago, Milwaukee and Puget Sound Railroad for its own use. The aggregate berthing space is 14 500 lin. ft. The business of the port has been greatly promoted by the location there of the principal shipping facilities of two important railway systems, and it now crowds closely on that of Seattle.

There are five other ports on the Sound worthy of particular note. These are Olympia, at the extreme southern end, and formerly important as the first point where travel from the Columbia touched the Sound—a very important circumstance before there were any good roads or railroads in the country. Everett is 30 miles north of Seattle, and is at the point where the Great Northern Railway reaches tidewater. It lies at the mouth of the Snohomish River, and is a port of considerable importance. Bellingham is another of the important secondary ports, and the farthest north of those considered in the paper. Anacortes is a flourishing port on Fidalgo Island reached by a branch line of the Great Northern. Port Townsend is the chief port on the Olympic Peninsula, and has a peculiar importance quite apart from its commercial business, which is slight. It stands at the gateway to Puget Sound, where the great defenses are, it is the headquarters of the Customs District of the Sound, the location of the Quarantine Station and Marine Hospital, the base for nearly all marine reckonings pertaining to the Sound, and, in other respects, an artificially important port. Its location is such, however, that, despite its excellent harbor, there is no reason to expect that it will ever develop an important commerce of its own.

The expenditures, in round numbers, already made or authorized, by the Government in improving the several harbors of the Sound, are as follows. There have also been local expenditures in most of them, but these amounts, except in the case of Seattle, are difficult to determine.

Olympia	\$160 000
Tacoma	320 000
Seattle (Canal)	2 750 000
Everett	700 000
Bellingham	60 000
Total	\$3 990 000

The commercial statistics of the Sound ports are given in Table 5, the totals being as follows:

Port.	Short Tons.	Valuation.
Seattle	2 371 628	$96 048 996
Tacoma	1 299 206	61 315 442
Other ports	2 244 889	28 892 495
Entire Sound	5 915 723	$186 256 933

TABLE 5.—FREIGHT TRAFFIC, 1910.

Article.	Amount (customary units).	Amount, short tons.	Valuation.
PORT OF SEATTLE.*			
Lumber	100 795 173 ft. b. m.	201 590	$1 562 567
Grain	1 675 745 bushels.	50 272	1 501 595
Flour	616 310 bbl.	60 398	2 619 887
Fodder		30 109	722 925
Fruit	148 219 cases.	3 705	256 072
Coal		637 851	2 706 147
Building material		†110 292	1 475 480
Merchandise		1 063 155	55 522 586
Oil	†616 133 bbl.	123 227	2 372 949
Miscellaneous		91 029	27 308 788
	Total	2 371 628	$96 048 996
PORT OF TACOMA.			
Lumber	268 221 170 ft. b. m.	404 427	$3 087 293
Grain		137 412	4 056 344
Flour		125 589	5 084 121
Fodder		19 747	271 730
Fruit		4 918	497 400
Coal		49 565	151 292
Oil	303 890 bbl.	60 778	528 728
Building material		163 022	1 008 541
Merchandise		170 473	31 290 942
Miscellaneous		163 275	15 339 051
	Total	1 299 206	$61 315 442
PUGET SOUND, SEATTLE AND TACOMA EXCLUDED.			
Lumber	1 196 745 204 ft. b. m.	1 686 207	$10 503 009
Grain		23 489	626 875
Flour		4 283	133 621
Fodder		12 705	234 012
Fruit		9 021	542 393
Coal		16 431	67 793
Oil	15 410 bbl.	3 142	24 527
Building material		178 230	537 253
Merchandise		136 511	10 084 509
Miscellaneous		174 870	6 138 503
	Total	2 244 889	$28 892 495

* Compiled from the Harbor Master's Report.
† Computed in part.

Vancouver, B. C.—The chief Canadian port, Vancouver, B. C., is in the vicinity of the mouth of the Great Fraser River, though actually not in the valley of that stream. More fortunate than Portland, Vancouver has been able to flank a troublesome river with its floods and shoals while still taking advantage of its valley on its rail route through the mountains. The port has been built in a land-locked bay called Burrard Inlet, surrounded on nearly all sides by towering hills which give the location a most beautiful setting. The area of the entire inlet inside the entrance is 23.6 miles, with a shore line of upward of 50 miles; but the port proper, as at present developed, utilizes only a small fraction of this. The shores are mainly of rock formation; the tidal range is 11 ft. mean and 16.4 ft. extreme. There is no ice, and the harbor is perfectly sheltered and has good anchorage.

The port facilities, as developed so far, are mainly on the south side of the east 5.5 miles of the Inlet. The portion of the Inlet comprising the harbor is about 1⅓ miles in width, connecting with the Gulf of Georgia on the west and the remainder of the Inlet on the east by two narrow channels designated First and Second Narrows, respectively. These Narrows were formed by the deposit of gravel, sand, and small boulders brought down by mountain streams on the north side of the Inlet and deposited in the shape of extensive bars. The Narrows are deep, but only about 500 ft. in width. Navigation through them is seriously handicapped by strong tidal currents, which, at the First Narrows, reach as high as 8 knots per hour at spring tide, and are such a menace to navigation that the Dominion Government has undertaken to widen the entrance by dredging to 1 200 ft. The improvement will not affect the current perceptibly, but will give more leeway for the navigation of ships. In most portions of the harbor the tidal currents interfere seriously with the berthing of ships.

New Westminster.—Ten miles south of Vancouver on the Fraser River, and 16 miles above its mouth, is the Town of New Westminster, which is taking active steps to develop port facilities for deep-sea shipping. The Fraser is a large river, not unlike the Columbia, but with a channel less difficult to improve and maintain. The ordinary tidal range at the port is only 5 ft., but the range between high and low water in the river is 14 ft. Reference will be made in another place to the steps being taken to develop this port.

Victoria.—This has been an important port in the past, and is so still, though its commercial business is not large. It is the chief port of Vancouver Island and on it is located the capital of British Columbia. The commercial port is on a small land-locked inlet with bald rocky shores. A few miles to the west is the celebrated Esquimault, an excellent port, strongly defended, and, prior to 1905, the headquarters of the British North Pacific Naval Squadron.

Prince Rupert.—This harbor is on an island within a well-sheltered inlet on the "inside passage" to Alaska, a little north of the mouth of Skeena River, the valley of which stream is the route of the western division of the railroad. The area of the entire inlet in front of and back of the island is about 27 sq. miles, and that of the shore line is about 80 miles. The tidal range is large, the maximum being 27 ft. and the mean about 14. The harbor is perfectly sheltered and is free from ice.

The port is very new, and its development has only begun. It now has about 2 700 ft. of berthing space, of which 600 ft. belongs to the Provincial Government and the rest to the Grand Trunk Pacific. It has a sectional dry dock 600 ft. long. The newness of the port and the fact that rail connection east is not yet opened, give little as yet for comparison with other ports. Its record lies all in the future.

The promoters of this important terminal enterprise adopted the almost unique course in civic growth of laying out their city before its occupancy by settlers began. It has been scientifically planned by artists of national reputation, and, if the plans are followed, it will be spared the disfiguration characteristic of most American cities.

III.—Notable Engineering Works.

The development of the ports of the Pacific has evolved a great range of engineering problems, some of which are of unusual magnitude, difficulty, and cost. A brief reference to the more important will be given. They include:

(1) Rock removal.

(2) Breakwater construction.

(3) Jetty construction.

(4) Lock construction.

(5) Dredging.

(6) Sea-wall and pier construction.

Rock Removal.—This class of work has been confined mainly to San Francisco Harbor and to ports north of the Strait of Juan de Fuca. In San Francisco the removals have been almost entirely of isolated rocks, some of which did not originally come to the surface. Another example of sunken rock removal is that in Roche Harbor, San Juan Island, Washington. Ledge rock removal has been and is being extensively carried on in British Columbia ports.

The various rock removals in the vicinity of San Francisco include the following: Blossom Rock, directly in the channel leading from Golden Gate into the harbor; Rincon Rock, now within the pier-head line and covered with a wharf; Arch Rock; the two Shag rocks; Centissima Rock in the Bonita Channel just outside the Golden Gate; and Noonday Rock, 33 miles at sea opposite the harbor entrance. Of these removals, Blossom Rock was most important. The method of removal originated with General Alexander of the U. S. Corps of Engineers, and the plans were very carefully worked out. They included the building of a coffer-dam on top of the rock, which rose to within 6 ft. of low water, unwatering the coffer-dam, sinking a shaft into the bed of the rock, and then excavating an immense chamber into which was to be placed the explosive. It was expected that the strong tidal currents would remove most of the shattered rock, and that the balance would be scraped into deep water. The contractor sank and securely fastened to the rock a timber crib which served the double purpose of a platform and a protection to an iron tube 6 ft. in diameter and 14 ft. high, which was placed through the center of the crib. A water-tight connection was effected with the rock by the use of a liberal quantity of cement. The tube was then unwatered and a shaft sunk to a reference 30 ft. below low water. During the progress of sinking the shaft so much leakage occurred through the rock that it was found expedient at two different times to telescope smaller tubes into the shaft and secure water-tightness by cementing the annular space between the rock sides of the shaft and the tubes. At a depth of 14.5 ft. into the rock the leakage became nominal, and the chamber was then easily excavated. The floor of the chamber was at reference 31.5 ft. below low tide, the axes being 135 and 55 ft., respectively. The roof conformed to the general shape of the surface of the rock and had a thickness varying between 14 and 18 ft. Four stone pillars were left at the shaft and timber supports were distributed throughout the chamber.

The explosive used was nitrate of soda, of which 4 300 lb. was placed in 38 wooden, ale barrels and in 7 old, boiler-iron tanks. The barrels were distributed around the edge of the chamber, and the tanks were scattered at about equal intervals on the floor. The blast took place on April 23d, 1870. The explosion was a pronounced success, but the chamber had not been made large enough to engulf the broken-up roof, and the hope that the current would remove the débris was not realized. The subsequent removal of débris, by scraping the blasting, cost as much as all the previous operations, up to and including the blast.

A contract for removing an additional 6 ft. from Blossom Rock in order to give a 30-ft. depth at low water, was let in November, 1902, for $45 142. The quantity of rock to be moved was 4 630 cu. yd. The contractor drilled a few holes, but abandoned the process for surface blasting and dredging the broken-up rock into deep water. The official reports do not give the reasons for the adoption of this method, but it may be conjectured that at least one reason was the shattered state of the rock due to former operations.

On January 2d, 1863, the clipper ship, *Noonday,* was wrecked on an uncharted rock in the Pacific opposite Golden Gate, and a demand arose for its removal. A search for the rock was made by the U. S. Coast Survey, and a pinnacle 5 ft. square on top, with 19½ ft. of water over it at low tide, was found and buoyed. In 1874 a contract was awarded to Mr. Edward Moore, of Portland, Me., for removing the menace to navigation, at an upset price of $20 000. Being in the open sea and subject to adverse weather conditions, the work entailed more hazard than similar work in a sheltered harbor. At first it was proposed to sink an encircling chain to a depth of 50 or 55 ft., at low tide, to which were to be attached packages of nitro-glycerine, but when operations were commenced the diver discovered below the required depth a cavity 10 ft. deep in one side of the rock. The plan was altered immediately to put the entire charge of explosive in the cavity. The charge was fired May 7th, 1875, and proved an entire success.

The method of subaqueous rock removal known as "bulldozing" apparently had its origin in San Francisco. It consists simply in placing high explosives in contact with rock surface and exploding it there without drilling or tamping. The shattered surface is then scraped away and the process repeated. While the use of explosives

by this method is excessive, the expense of drilling is saved. The results, so far, however, do not seem to commend the method on the score of either efficiency or cost. In 1907-08 a submerged rock at the entrance to Roche Harbor, Puget Sound, was removed by this method. The volume of the portion to be removed was 1 500 cu. yd., and the contract price was $23 460; but the work probably cost the contractor more than this. The official Government report says that "the method, as applied in this case, did not prove satisfactory, and the contractors state that if they had the work to do over they would drill the rock."

In the work in San Francisco Bay, where the masses to be removed were considerable, the contract price per cubic yard was under $10, as, for example, the work on Rincon Rock, before it was abandoned, cost $9.06 per cu. yd. for the removal of 4 357 cu. yd.; and the cost of removing 32 000 cu. yd. on the Shag and Arch Rocks was $7.93 per cu. yd.

The subaqueous basaltic rock excavation at the eastern end of Victoria Harbor was performed by the drilling method. The single drill was mounted on a platform float that could be raised clear of high tide by steam power spuds. Steam was furnished from boilers on a barge. Thirty-two holes were drilled at one setting of the raised platform. The holes were 2.5 in. in diameter, drilled to a depth of 2.5 ft. below grade (20 ft. at low water), and spaced 3 by 3 ft. The actual cost for drilling and blasting only was $6.17 per cu. yd., without interest, depreciation, or plant renewal.

The blasting in Vancouver Harbor will be a more delicate operation, as it will be carried on in close proximity to constructed piers. Holes 3 in. in diameter, spaced 5 by 5½ ft. centers, will be drilled to a depth of 3 ft. below the required grade line (36 ft. at low tide), and shot off in series. It is expected that 1½ lb. of dynamite per foot of hole will be used. The estimated cost for drilling and blasting is $5.60 per cu. yd. Dredging of the broken rock will approximate an additional $1 per cu. yd. The Vancouver rock is a sloping sandstone ledge overlaid with from 3 to 7 ft. of gravel, hardpan, and disintegrated sand rock.

Breakwater Construction.—As an example of breakwater construction strictly speaking—that is, work done for the primary purpose of protecting anchorage areas or harbor entrances from ocean storms—the San Pedro Breakwater at Port Los Angeles is the most important

on the Coast. As finally constructed, it has a length of 11 000 ft. and extends first on a curve and then in a straight line easterly from Point Fermin, thus giving shelter against south and southwest exposure and in particular completely protecting the entrance to the inner harbor which is behind the inner end of the structure. The breakwater is built entirely of rock, except a concrete block 40 ft. square and 20 ft. high at the outer end. The base of this block is 3 ft. below the low-water plane. The construction of the breakwater was effected from a double trestle carrying tracks of standard gauge. The substructure was built to the elevation of low water and consists of a central portion of stone weighing 1 ton or less, with larger stones on the side slopes. The superstructure is 38 ft. wide at the bottom, and 20 ft. wide at the top (14 ft. above low water), with stepped sides of huge blocks of stone placed by cranes. On the ocean side the stones weigh 16 000 lb. or more; on the harbor side, 6 000 lb. or more. The space between the sides is compactly filled with rock of all sizes, and the top is finished with large stones. The side blocks have been placed with such care that they seldom depart more than 6 in. from exact position.

Jetty Construction.—As used in this connection, the term, jetty, applies to a structure designed to serve the primary purpose of concentrating and directing ocean currents to develop scouring effects over shoal bars, and the secondary purpose of protecting such crossing from storms as far as their location for the first purpose will permit. They are, therefore, a combination of training dike and breakwater. There are four prominent examples on that portion of the Pacific Coast considered in this paper, all of which are north of San Francisco, namely, Humboldt Bay, Coos Bay, the Columbia River, and Grays Harbor. There are also numerous minor examples, as at San Diego, Los Angeles, and Oakland. The principles involved are essentially the same in all, though varied, of course, in application to suit the conditions of each case. The Columbia Jetty far exceeds any of the others in importance and in the magnitude of the work, and the description will be limited to this one example.

As originally projected, the jetty extends in a west-by-northwest direction 4½ miles from the east side of Point Adams across Clatsop Spit and out to deep water. The jetty was to be built of large size stone on a brush mattress and raised to the level of low tide. Later,

the project was changed to raise it to high tide, and four groins were to be built from the north side to arrest scour. Work was begun in 1885 under an estimate of $3 710 000, and was completed ten years later at half the estimate. The depth over the bar had increased, in the meanwhile, from 21 to 31 ft., and the work seemed to have accomplished its purpose perfectly.

Unfortunately, the bar began rebuilding itself, the depths shoaled, and several years later there was only the original depth of 21 ft., and it seemed to be necessary to start all over. A new project was adopted extending the jetty 3 miles farther, or somewhat beyond the longitude of Cape Disappointment on the north shore. The work was to be supplemented by dredging over the bar. This extension is now nearing completion and has resulted in a present depth of from 25 to 27 ft. A permanent depth of 30 ft. is sought.

This work, it is scarcely necessary to remark, is probably the largest and most difficult of its kind ever attempted, and has evolved some of the most ingenious and elaborate methods for overcoming difficulties in the whole range of engineering. The two great obstacles to be overcome in the execution of the work are the storms and the *teredo*, and it is the fact of the storm that gives the *teredo* its "inning." The roughness of the sea precludes the use of barges for dumping rock, thus necessitating a trestle, and the trestle piling is the particular delight of the *teredo* which puts it out of commission in from 10 to 20 months. The rebuilding of the trestle on the rock of the jetty, which has been repeatedly necessary, and the methods resorted to in order to arrest the work of the *teredo*, have taxed the resources of the engineers to the utmost.

The jetty itself, as now being built, consists of a bed course of small rock as a substitute for brush mattress, it being found impracticable to use the mattress beyond the shoal depths of Clatsop Spit. On the foundation so provided heavier rock is deposited, and the whole is covered on the seaward slope with very heavy rock ranging in weight from 6 to 16 tons per piece. Even these enormous masses are displaced by the storms, and the embankment is progressively shaken down during each winter season.

The long continuance of this work has evolved a very efficient system for its management—one which it would be difficult perhaps to improve on, because every feature is the result of actual experience.

The pile-driver is a study in itself. The support of the pile trestle on its bed of huge boulders, the arrangement of the movable car beds and of the cars themselves for the quick handling of the rock, the systematic organization of the work which moves with the regularity of clockwork, and, finally, the rational division of the work between the contract and day-labor systems, assigning to each the share it is best adapted to carry out—all combine to make this great work well worthy the attention of the engineers of the United States.*

Lock Construction.—Locks are an important adjunct of port development in English ports where they are applied to overcome the drawback of excessive tidal fluctuation. They are nowhere so used on the American Continent, but they are about to be used in the Port of Seattle for the purpose of effecting a very large extension of the harbor, as previously explained. The lock which separates the main harbor from the Sound is located at the outlet of Salmon Bay, 1 mile inland from deep water, at a sheltered and easily defended point called the Narrows.

The lock is really a double structure, comprising a large chamber for heavy shipping, log rafts, etc., and a small chamber for the use of the mosquito fleet. It is certain that small-boat traffic through the canal will be large, and the plans provide a small, quick-operating lock for its use so as to avoid the slower and more expensive operations of the larger lock. It is considered that this provision will prove one of the most useful features of the canal. A similar arrangement is provided on the Manchester Ship Canal, in England. It seems to the writer that if the Panama Canal plans are open to any important criticism it is in not providing a flight of smaller locks. To operate those immense locks for the passage of small boats cannot fail to prove an immense waste of energy and time and possibly an interference with traffic.

Returning to the Lake Washington Canal lock, a description of that structure, which is now in process of erection, will be given. The larger lock will be 825 ft. long and 80 ft. wide, or large enough to admit without breaking the large log rafts which are certain to constitute an important part of the traffic. The sill of the lower gate will be at reference 25 ft. below extreme low tide, 29 ft. below

* Those who desire a detailed description are referred to an article on page 109 of *Engineering News* of July 30th, 1908.

mean lower low tide, and 35.67 ft. below mean sea level. The sill to the upper gates will be 36 ft. below the low-water level of the canal and 38 ft. below the high-water level. The lock will be supplied with intermediate gates which may be used to divide the chamber into lengths of 350 and 475 ft., respectively, for convenience and economy of water in the passage of shorter vessels. There will be guard gates at the two entrances, and a movable dam at the upper end for emergencies arising from accidents. The small lock will be built on the south side of the large lock, and will be 150 ft. by 30 ft. and 16 ft. deep over the miter sills.

The lock walls will be of mass concrete, gravity section, straight-front face, with a toe projecting into the lock chamber, and stepped back. The filling culverts will be of large elliptical section running the full length of both walls and connecting with the chamber by numerous side culverts at right angles to the axis of the lock. The controlling culvert gates will be of the Stoney type in the large lock and of some form of cylindrical balanced valves in the small one. It is expected to fill or empty the chamber of the big lock in 8 min.

The lock gates will be of steel, parallel straight faces except near the front and back edges. All gates will be provided with air chambers to relieve the strain on anchorages and to permit of floating the guard gates out of position for repairs and painting. The main gates will be reached by closing the guard gates and pumping out the chamber.

The most interesting detail of the lock, and the one on which there will be most divergence of opinion, is the lock floor. The ever-recurring debatable question in lock construction arises here—shall the floor be made strong enough to resist an upward hydrostatic pressure or shall it be a mere pavement to protect the underlying material from scour, reliance being placed on the cut-off walls to prevent leakage? The lock floor is not yet determined on, the decision being held in abeyance until an examination can be made of the foundation material, when the lock coffer-dam is unwatered.

The lock when completed will be exceeded in size, on the American Continent, by the Panama Lock only; it will be on a par with the new Sault Lock, greater in some particulars, but less in others, and will entail the solution of more difficult engineering problems than the one at the Sault. It will rank with the greatest European locks and take

its place in shipping and engineering circles as one of the very prominent structures of its class.

The principal data relating to the canal are as follows:

Extreme low tide in Puget Sound......	Reference	0	
Mean lower low tide, “ “	“	4	
Mean low tide, “ “	“	6.89	
Mean sea level, “ “	“	10.67	
Mean high tide, “ “	“	14.50	
Mean higher high tide, “ “	“	15.23	
Extreme high tide, “ “	“	18	
Present level of Lake Union...........	“	25	
“ low-water level in Lake Washington	“	33	
“ high-water level in Lake Washington	“	38	
Proposed low-water level in canal.......	“	24	
“ high- “ “ “ “	“	26	
Length of large lock between quoins...........		825	ft.
Width “ “ “ “ “		80	“
Depth on lower sill at extreme low tide..........		25	“
“ “ “ “ “ mean sea level...........		35.67	“
“ “ upper “ “ low water................		36.00	“
“ “ “ “ “ high “		38.00	“
Lift of lock from extreme low tide to high water...		26.00	“
“ “ “ “ “ high “ “ low “ ...		6.00	“
Depth of water in main body of Lake Union......		40.00	“
“ “ “ “ Lake Washington		40.00 to 200.00	“
Length of canal from deep water in Puget Sound to deep water in Lake Washington..........		8.00	miles

Dredging.—Pacific Coast ports are no exception to the almost universal need of dredging operations to secure full navigable depths in channels and docks. Seattle and Tacoma come nearest to being exceptions, but even in these ports waterways have been created artificially by dredging. At Seattle, for example, a great deal of work has been done in reclaiming the tide flats of the Duwamish Delta, and creating artificial waterways there, while now extensive operations are in progress in excavating the Duwamish Waterway and the Lake

Washington Canal. None of these operations was necessary, however, in the main harbor, which everywhere had ample depth. At the other extreme is Los Angeles, where the harbor itself is being built almost entirely from excavation. Differing from either of these is Portland, where the channel of approach to the harbor requires very extensive and continuous dredging, not only for original deepening, but for maintenance of depth. San Diego, Oakland, San Pablo Bay, Grays Harbor, and several of the ports in Puget Sound, have been thus improved by excavation. As a rule, this work has been done with Government appropriations, though in some ports, such as Portland, and, more recently, Los Angeles and Seattle, local funds are being contributed to the work. Now that the reclamation of tide flats is becoming so profitable an enterprise, dredging will be resorted to more than ever, the operation serving the double purpose of excavating slips and channels and filling the abutting lands.

The plant used in this work embraces every practical variety of dredging machinery, from the heaviest sea-going hopper or suction dredge to ordinary steam shovels loaded on scows. The Pacific Coast dredging fleet, taken as a whole, both publicly and privately owned, is a formidable one. In the more localized operations electricity is sometimes used as the motive power. Oil is extensively used as fuel. All varieties of excavation are encountered, from the lightest alluvium to indurated clays that border closely on soft rock. In California great use is made of the clamshell dredge, with enormously long booms, designed for levee building on the Lower Sacramento and San Joaquin Rivers, and on some tributaries of the larger stream great hydraulic dredges are used to glean out the particles of gold which the improvident miner wasted as he sluiced down the auriferous hillsides a generation and more ago.

It is needless to describe in any detail these dredging operations, for they present nothing sufficiently characteristic to mark them from those with which engineers are familiar in the East. It will, however, be of interest to refer briefly to two of the dredges. The latest sea-going hopper dredge built by the United States Government is the *Chinook* for use on the Columbia River Bar. It is a converted transport, 445 ft. long, of 5 590 gross tonnage. The transformation was made in 1902, but many alterations and repairs have been made since that time. The dredge has two 20-in. pumps and two

bins of a total capacity of 3 600 cu. yd. Its performance during the fiscal year ending June 30th, 1911, was:

Days worked	160
Cubic yards excavated	722 431
Total operating cost	$58 943.15
Cost per cubic yard	$ 0.08159

Probably the most effective dredging plant for all purposes on the Pacific Coast is the *Frühling,* built in Germany in 1906 at a cost of $275 000 exclusive of duty and cost of steaming to British Columbia. It is the property of the Department of Public Works, of Canada, and operates on the Fraser River. It is of the "Frühling" scraper suction type, elevating material by a pair of 16-in. centrifugal pumps through pipes and delivering to hoppers inside the hull. It is a sea-going dredge, of steel construction throughout, and is self-propelled by twin screws. Its length between perpendiculars is 187 ft.; the extreme beam is 34.5 ft., the draft, loaded, about 14 ft., and the hopper capacity at this draft, 800 cu. yd. Its performance from April 1st, 1910, to March 31st, 1911, was as follows:

Hours worked	862.5
Cubic yards excavated	924 800
Average cubic yards per hour	1 072
Average cubic yards per day of 9 hours, including all delays	3 000
Maintenance cost	$37 948.10
Repairs cost	8 835.58
Total cost	46 783.68
Cost per cubic yard	$0.0505

Sea-Wall and Pier Construction.—As stated elsewhere in this paper, there has been very little massive sea-wall construction in the ports of this Coast, San Francisco being the only considerable exception. The earlier type of cross-section adopted there consisted of a loose rock core faced with heavy stone paving. More recently, reinforced concrete on a rubble and pile foundation is being used. There have been some attempts to build vertical concrete walls in Seattle Harbor, but the excessive cost and the injurious effect of the sea-water on the concrete have not led to their repetition. Where the tidal range

is as great as it is at Seattle, a sea wall has to stand above the foundation fully 60 ft., and the cost of a wall built on any adequate plan is almost certain to exceed $300 per lin. ft. This condition has led to various schemes for supporting the land face without the cost of a wall. Creosote or concrete piling, concrete piers, or cylinders supporting a platform back for a distance of from 30 to 60 ft., afford an opportunity to carry the slope up at an angle which can be made stable at moderate expense with brush or rock paving or a combination of both. Much thought is now being given to this problem, and a type of construction will doubtless be evolved, which will be a fair compromise between cost and durability.

The same difficulties enter the problem of pier construction. The method heretofore in force along the Coast has been mainly that of wooden piling and timber superstructures—in fact the simplest form of practical construction. Latterly, there has been a strong wave of sentiment for "permanent" work, which means reinforced concrete substructures and fire-proof superstructures. Conservative considerations, however, are going to place a strict limit on the application of this method, and timber construction, with creosoted piling, where subject to the ravages of the *teredo,* will continue to be the rule. Any extensive adaptation of the costly masonry construction of European ports is not to be expected.

The serious problem is, of course, that of the substructure. Ordinary piling will not last a year in these waters. Creosote piling, if of superior quality, is safe for fifteen years; but the lack of stability in very deep water, where depth necessary for draft is augmented by a high tidal range; the great cost of replacement, requiring, as it often does, the removal of shed roofs and the suspension of business; the increased danger from incendiarism, rats, theft, etc., and draft in conflagration, all due to the open work underneath, are very serious drawbacks which go far to offset the advantages of less cost. The most practical compromise with timber construction would seem to be creosote piling under wharf platforms with protected slopes to bulkheads on line with the shed walls, thus giving solid fills under the sheds. Concrete piling or concrete cylindrical piers, such as are used in the splendid new Government piers at Fort Mason, San Francisco Bay, are, of course, more to be desired for this exterior work. A brief description will be given of two docks, one a fine example of

typical timber construction and the other an original plan of solving the difficulties referred to.

The Grand Trunk Pacific Dock at the foot of Madison Street, Seattle, is a recent structure. It is 625 ft. long, 128 ft. wide, and is supported entirely on creosoted piles, the length of the bearing piles varying from 55 to 90 ft., with bracing piles as long as 110 ft. The bents are 10 ft. apart, and the piles in the bents are spaced from 6 ft. centers at the inner end to 3 ft. at the center. The capping and floor systems are of very thorough construction, and the dock appears to have thorough stability, notwithstanding the great depth of water in which it stands.

The main shed is 90 ft. wide, with a platform 8 ft. wide on the south side and 16 ft. on the north, the latter carrying a railroad track. There are five adjustable slips on each side and one at the outer end worked by worm gear. The fender piles are of tight-bark fir and are driven two at every bent and fastened to 12 by 12-in. stringers with ¾-in. straps. At the outer end they are spaced 3 ft. apart, and clusters of seven piles are driven at the outer corners. The timber used in construction was 241 775 lin. ft. of creosoted piling, 71 000 ft. B. M. of creosoted bracing, and 1 113 000 ft. B. M. of other lumber. In its finish, offices, equipment, and general lay-out, it is the best example of dock construction on Puget Sound.

The second of the docks selected for a brief description here is the Great Northern Dock at Vancouver, B. C. It was designed by A. W. Münster, M. Am. Soc. C. E. The pier is 450 ft. long by 293 ft. wide, with a berthing space of 30-ft. depth at low water on each side, and with two sheds, 403 by 102 ft. Between the buildings is a space 41 ft. wide. The distinctive feature of the structure is a cantilever platform supported on cylindrical concrete piers, which themselves rest on a sandstone ledge. These piers are 4.5 ft. in diameter, with a base enlarged to 10 ft. The foundation is carried down into the hardpan or sandstone ledge to a minimum depth of 3 ft. below the bottom of the slip. The piers will have an average height of 52 ft. On top of them will be placed a longitudinal concrete girder, 3 ft. wide and about 7 ft. in depth, which will carry the transverse girders forming the immediate support of the floor slab, which are placed 12.5 ft. center to center. The transverse girders are from 2 to 3 ft. wide, 6 ft. deep, and 51 ft. long. They will project 16.5 ft. beyond the line of

the piers and, acting as a cantilever, will carry corresponding parts of the front platform. The inner end of these girders will rest on a line of 16-in. concrete piles driven to refusal and stayed in position before the rock embankment is placed. The floor slab is to be 7 in. thick. The concrete platform and beams have been designed for a distributed load of 500 lb. per sq. ft. The railroad track running the length of the platform is placed centrally over the longitudinal girders. The reinforced concrete construction supporting the track is proportioned to carry a 100-ton switch engine. Tie-rods encased in concrete will anchor the concrete platform firmly to the embankment.

The warehouse buildings are of wooden construction covered with galvanized iron. The roof is to be carried on trusses spanning the whole width of the building, leaving an unobstructed floor space. The floor inside of the building will be paved with creosoted blocks on a concrete base. The concrete slabs on the outside platform are to be covered with an asphalt wearing surface. The cost of the completed dock and buildings will be approximately $1.50 per sq. ft.

IV.—Administrative Systems.

Public Ownership and Control.—In discussing the different systems of administrative control which obtain in the ports of the Pacific Coast, it is necessary to take cognizance of the present strong trend of public thought in favor of public ownership and control of port facilities. It would be interesting to examine the reasons which underlie this movement, but it must suffice for the present purpose to say that they are essentially the same as those that have substituted the free public highway for the toll road of former times. For the wharf is in reality the transfer point between a free public highway on the land and a free public highway on the water, and there is no more reason why this point of transfer should be made a subject of private monopoly than either highway. The transfer may not indeed be made wholly free, and the user may properly pay the actual cost of use, just as a user of the public highways pays another for hauling himself and goods along it when he does not perform that service himself. Moreover, a public wharf being built and maintained through the agency of only a limited portion of the community actually using it, the cost cannot be equitably charged to that portion alone, as it would be if absolutely free of tolls and maintained

solely by taxation. A strictly free port is not, therefore, one of the things to be expected or desired, and is not likely to be realized except as a result of cut-throat competition between rival ports. The writer will not go further into this extremely important matter of port control than to repeat a summary of the principles involved as stated by him in a recently published article.*

"The true policy for a great port to pursue in these matters would, therefore, seem in the light of present-day advanced practice, to be essentially as follows:

"The wharves with the buildings and appurtenances directly connected therewith should be built and owned by the port itself, and should be operated either directly or by short-term assignments in such way as to eliminate entirely the element of private monopoly, and to bring the cost of service to the lowest point consistent with maintenance, operation, and gradual recoupment of original outlay. Even when this return cannot be fully realized and the utilities have to be supported in part by taxation, public ownership may still be vastly advantageous through indirect benefits, just as are public highways which yield no direct revenue whatever.

"The system of railway tracks serving the wharves should be consolidated in the larger ports, under a single unified management, separate and distinct from the tributary railway systems, except as they form the connecting link between such railways and the wharves. These service tracks may be managed either by a private terminal company, subject to port supervision, or by the port authorities themselves. The latter method has definite advantages.

"So far as a port may directly or indirectly foster industrial development in connection with its own utilities, it should do so upon terms which will attract private capital and insure it a sufficient lease of life to enable it to work out its plans and enjoy the fruition thereof. The case is essentially different from that of public utilities, and so far as a port may be drawn into such enterprises it must necessarily be upon a different basis. There is no question that many industrial enterprises have so close a connection with water and rail transfer facilities as to make them quasi-public enterprises. While they are not of a character which a port will ordinarily take up as an original proposition, they are entitled to every practicable measure of co-operation and assistance which the port may give, and are to be considered as an essential element in its successful development."

In many European ports there is a close approach to this ideal, though with wide variations under different governments and customs.

* *Railway and Marine News*, Seattle, Wash., May 1st, 1912.

In America the ultra-democratic ideal of *laissez faire* and non-interference by Government led to an almost universal custom of private ownership of harbor frontages. Astute business foresight was not slow to appreciate the opportunity thus offered, and the important points of every important harbor, almost in its infancy, became private monopolies. When the railroad came, this evil assumed an aggravated form, and it would be a threadbare story to describe the extensive appropriation of water frontage by the railroads, particularly in the newer sections of the country.

As time has passed and the public has been brought face to face with a situation resulting from its own earlier indifference, a change in sentiment has taken place and is just now in full tide of progress. The people are determined to recover the birthright which they bartered away for a paltry mess of pottage. It is no paltry ransom, however, that they are being compelled to pay. Untold millions are being paid and will continue to be paid for what should never have been parted with at all; but the people are determined, cost what it may, to regain control of their water fronts. Private interests resist—it is the loss of a rich monopoly—but they cannot prevail, and the force of public sentiment should be brought to bear to make the struggle as brief as possible.

San Francisco.—Among the ports of the Pacific Coast, San Francisco is the one bright exception in the past to the rule of private ownership. The principle of public ownership and control has, indeed, been adopted in all three of the Coast States; but it has failed utterly in application except in the Port of San Francisco. A significant proof of the overshadowing importance of this port in the past history of the State of California is seen in the fact that early legislation on port matters was directed almost exclusively to San Francisco, and a system of government was established there which was not applied to any other port of the State. The statute which created the Board of State Harbor Commissioners (note, it is a State tribunal, perfectly general in its title) limits its jurisdiction to "possession and control of that portion of the Bay of San Francisco" (description by metes and bounds takes up three pages in the printed statute), extending in general terms from the Federal Reservation (the Presidio) near Golden Gate around into the Bay to the line between San Francisco and San Mateo Counties. Though a State body, its

activities are purely local and pertain exclusively to a portion of the water-front of a single city of the State. Its officers are appointed by the Governor and confirmed by the Senate, and are a regular part of the political machinery of the State. The first Board of Commissioners was appointed in 1863, and the Board has been in existence, therefore, for very nearly half a century.

Whatever may have been the failings of this body—and it has been accused of many things, particularly of being a subservient political organization—the fact that, through all these years, in a State where railroad influence was supreme, it has maintained the integrity of the water-front of San Francisco and has kept it in public control, must stand to its everlasting credit. To-day San Francisco, New Orleans, and Montreal are the only American ports in which the principle of public ownership is practically applied on a large scale, and while the system in force in San Francisco is not as perfect as that in New Orleans, still it is a vast improvement over that which prevails generally in the ports of this country.

The vicissitudes of economic progress, however, have at last made the system of State control antiquated, and San Francisco is now ready to forego the pupilage of the State and take her future into her own hands. She wants home rule. She notes with anxious eye the prodigious strides of Los Angeles, San Diego, and particularly the cities across her own Bay. These ports are helping themselves, except that they are receiving from the State grants of tide lands amounting to many thousands of acres. They are organizing on the basis of their own needs. They issue their own bonds. They prepare their own plans. They fight their own battles. They are not tied down to legislative sanction for the measures they desire to adopt, and do not have to go to the electorate of the whole State for financial aid. They may be no longer willing to help pay San Francisco's bills, and it behooves San Francisco to get authority to pay them herself. She is waking up to the situation and is now earnestly agitating for complete home rule of her port. Undoubtedly it will come, and with it a more energetic and efficient working out of her destiny.

The small importance of the east side of the Bay in the early years made it an easy prey to private interests. Although the water-front of Oakland was granted to the town by the State as far back

as 1852, it immediately passed into private hands on a long-term lease, and finally fell into the control of the Southern Pacific, from which it has only recently been wrested by strenuous litigation. The port is fully alive to its opportunities, and the course of development from now on will unquestionably be in the direction of public ownership and control. As yet there is no port organization separate and distinct from the City Government. In time the east-side towns undoubtedly will consolidate, and a comprehensive port organization will be developed. Whether such an organization for the whole of San Francisco Bay will ever be adopted is doubtful. The jealousies of the two sides are too strong to make union practicable at present; but to the onlooker from the outside, a port commission for the whole of San Francisco Bay, with unified development of the entire port, would seem to be the best plan.

Los Angeles.—In Los Angeles there is no separate port organization distinct from the City, but the policy of public ownership is making great headway. Probably the bitterest struggle ever waged to rid a port of the incubus of private monopoly of its water-front is that of Los Angeles against the allied Southern Pacific and other interests. Of the final triumph of the people's cause there is no longer any doubt, though the delay and expense of litigation are a severe strain on the patience of the community. The administration of the port is divided on indefinite lines between a board of public works and a board of harbor commissioners. The method is, of course, unbusinesslike and has resulted, as might have been foreseen, in conflicts of authority which have seriously disturbed the orderly procedure of the work of port development. One of the first problems which Los Angeles should solve is the working out of an efficient system of port administration.

San Diego.—San Diego furnishes an interesting example of the prevalence of home-rule sentiment in Californian ports. In 1910 the State voted $1 500 000 in bonds for the development of the harbor; but the people preferred home rule, and the Legislature annulled the grant and ceded the tide lands within the city limits to the City on condition that the latter shall expend $1 000 000 in port development along specified lines and within a specified time. There is no separate port organization in San Diego, and the development work is carried out as an adjunct of the City Administration.

Portland.—The Oregon system of port management is admittedly not a satisfactory one, and, as exemplified by its application to the principal port of the State, it is in no sense an efficient one. It does not embrace any comprehensive scheme of port development and control, and, until quite recently, it contemplated nothing in the line of public ownership and control of dock facilities; it left the whole matter to private enterprise. The river front was originally deeded to the City, but has passed almost entirely into the hands of private interests—mainly that of the railroads. So strong was the hold of this system that, when the "Port of Portland" was created twenty years ago, its purpose was not at all to provide port facilities, much less acquire or control those in existence, but solely to secure and maintain a 25-ft. channel to the sea. It was simply an ally of the Government in its regular work of channel improvement. Certain specific functions have since been added, such as control of pilotage charges and the building of a floating dry dock; but in its essential purpose it is still an agency for channel improvement only. Its name is, therefore, a misnomer, and, when public sentiment became aroused recently to the necessity for some actual measures in port development, a new body was organized—the Portland Dock Commission—to take up this particular line of work. A bond issue of $2 500 000 was authorized, and plans are actively in progress for the construction of a system of public docks. On the face of it, the whole arrangement appears to be of an unsatisfactory and piecemeal order. Neither the original nor the new commission, nor even both together, has anything like a comprehensive jurisdiction; but their work has to be supplemented by co-operation with the City, and thus there is a sort of three-cornered jurisdiction of the port. It is scarcely to be expected that such a system can be permanently satisfactory. In all probability there will result a consolidated port organization, which will comprehend the present divided jurisdictions and proceed systematically with a unified upbuilding and administration of the port.

The Washington State Law.—The State of Washington has recently taken the most advanced position of any of the Pacific Coast States in this important matter of port administration. By Act of March 14th, 1911, it provided a system of comprehensive port development and control under an organization for that exclusive purpose. The organization is styled a Port Commission, and is composed of

three members elected for terms of one, two, and three years, respectively, in the first instance, and one Commissioner each year thereafter for the three-year term. The powers of the Commission are very comprehensive, embracing the whole range of port development and a large measure of port control. Certain powers are still lacking for the legal enforcement of rules, etc., and there is doubt as to the extent of the Commission's power in promoting industrial enterprises. There are, however, certain crude provisions which need modification to make the statute run smoothly; but on the whole it is the most elaborate, comprehensive, and effective measure of the kind ever passed. It will take time for it to gain full recognition, for it strikes at the root of private monopoly in communities where a large element of the so-called "business" interests is immovably committed to such monopoly; but, if it ever has half a chance to demonstrate its efficiency, it will command popular approval. Two districts have already been organized—one in King County, in which Seattle is located, and one in Chehalis County, which embraces Gray's Harbor. In the Seattle District the law has been put into effect with great vigor, and a programme of \$6 000 000 has been adopted by popular vote.

The development of the Port of Seattle prior to the passage of this law is a typical example of the complete miscarriage of a great purpose. In no State in the Union was a broader view taken of public questions of this character in framing the Constitution than in Washington. It was the purpose of the State to hold the tide lands in perpetuity for the use of the people, and the first Board of Harbor Line Commissioners was in earnest and determined to abide by the purpose of the Constitution. It would have been a great thing for Seattle if it had prevailed; but the time was not ripe. Public sentiment did not support it. It robbed private interests of too many opportunities for speculation. The false cry of business necessity prevailed, and in one way and another the Constitutional injunction was completely nullified, and the tide lands, now worth many millions, passed wholly into private hands at almost nominal rates. Streets were vacated, and even the harbor area, the phantom relic of a beneficent intention, was so tied up to abutting lands as to become a private monopoly. To-day the Port Commission, in trying to recover enough of this wasted heritage to build its first piers, is asked to pay, in some instances, as high as \$86 000 per acre.

British Columbian Ports.—The British Columbian ports, particularly Vancouver, are wholly private-ownership ports, and the water-front development is almost exclusively in the hands of the railroads. It is a striking contrast to the great Canadian port of Montreal, which is one of the most progressive public-ownership ports in the world. Vancouver is passing through the boom experience of most new localities in the West, and is willing to make almost any concession to private interests for what promises most for the immediate present. Not until after the present period of expansion has passed, and public thought, now absorbed in speculation activities, has time to take stock of the situation, is it likely that the problems of port control will be given very serious consideration.

Port Charges.—The diversity of administrative method among the Coast ports is particularly manifest in port charges. The widest difference prevails, and in only one port is there a definite and uniform system. San Francisco has a well-defined schedule of charges, and, whether the best adapted to the purpose or not, it is carefully worked out and serves its purpose very well. In most other ports of the Coast, rates are a matter of private arrangement, although in each there is generally a voluntary agreement among shippers as to what they shall be. They vary greatly, and even in the same port different rates are often applied according to the exigencies of the case. Thus, in Seattle, the basing rate on foreign cargoes is 25 cents per ton, and on coast and local cargoes 50 cents per ton. In San Francisco the whole matter is strictly regulated by a public body, and there is complete uniformity and absence of discrimination. Heretofore in Seattle there has been no such regulation except the ineffectual regulation of the State Commission, and there are frequent complaints of discrimination and irregularity. The two systems, moreover, are essentially different. In San Francisco there is a substantial dockage charge against the vessel; in Seattle there is no such charge, the entire charge being against the cargo. It is this custom which has given Seattle the reputation of being a "free port," but the freedom refers only to the vessel itself. The system is not an equitable one, and will have to be changed as soon as an agreement among the Sound ports can be reached. The San Francisco system is the more rational one in this respect. It comprises three fundamental charges, which have been defined officially as follows:

Dockage.—The charge made for vessels occupying berths at the wharves or moored in any slip or channel.

Tolls.—The charge made for merchandise passing over State premises.

Wharfage.—The charge made for leaving merchandise on State premises longer than the time specified in the rules.

In the Puget Sound ports there are no dockage charges unless a vessel is lying alongside a dock unengaged. The "handling" of the cargo corresponds to "tolls" in San Francisco, though it is less definite in its application and sometimes has no application at all.

While the San Francisco system appeals to the layman as the more rational and just, the ship operator is positive in his preference for the Puget Sound system (if it can be called such) and holds that it is decidedly the cheaper port. As far as the vessel is concerned this is true, but there is no great difference in amount where the total cost to ship and cargo is considered. Labor conditions in San Francisco make stevedoring higher than on Puget Sound, but this is a matter entirely apart from the port regulations in themselves. Labor unions may at any time force wages in other ports to the San Francisco scale, and, therefore, a correct comparison of systems should assume uniformity of wages.

In Los Angeles the charges are classified under the heads of dockage, handling, and wharfage, "handling" corresponding to "tolls" in the San Francisco schedule. The rates in San Francisco are fixed by the Board of State Harbor Commissioners, supplemented by several statutory enactments. In Los Angeles they are fixed by the Board of Harbor Commissioners, a branch of the City Government. The City charter gives this Board the same legal powers which the State Board enjoys in San Francisco in regard to the enforcement of its rates and regulations, and gives it the power to punish violations thereof. In San Diego there appears as yet to be no system of local public control of rates, all the docks being in the hands of private interests. As soon as the new municipal docks are completed, the control of rates will be reduced to a system.

In Portland, as in San Francisco, pilotage is compulsory and is an important item of the vessel's expense while in port. The wharves being all private property, there is no public control of rates, and the

charges of various kinds are determined largely by the individual wharf owners, although, of course, there is general conformity to an agreed schedule. The new Dock Commission, by the erection of its own wharves, will naturally assume control of future charges.

In Seattle the newly created Port Commission has the power to fix rates, but has exercised that power for the present only as a matter of form to comply with certain requirements of the statute. Prior to the actual completion of the docks which it contemplates building, it will adopt a thoroughly considered rate schedule. Under the present law it has no legal machinery with which to enforce compliance with its rules, but this defect will doubtless be corrected by the next Legislature.

In Tacoma and other Sound ports, practically the same system and schedule of charges prevail as at Seattle, but it may be expected that, in the near future, an effort will be made to revise the whole system through some mutual agreement.

The British Columbian ports being still almost exclusively under railroad control and ownership, as far as port facilities are concerned, the rates are privately controlled, and the charges are absorbed largely by the railroads.

Co-operation among Ports.—The writer's study of this question has led him to believe that important results might follow a duly organized conference among the recognized authorities of the various ports on the Coast with a view to evolving a uniform system of port administration and of port charges.

V.—Plans for the Future.

Let us now take a brief survey of the ports of the Coast from the viewpoint of their plans for the future. In the first part of this paper it was stated that the Coast is putting its house in order for the reception of the trade which will come from the opening of the Panama Canal. It is a safe assumption that nine-tenths of the funds provided in the past three years, or that will be provided in the next three, for the development of the Coast ports, have been secured through the argument that they are necessary for the above purpose. Ambitious plans have been prepared by recognized experts looking to a future a hundred years ahead and involving expenditures which can only be guessed at. To a great extent these plans will slumber indefinitely, because of

TABLE 6.—COMPARATIVE COSTS TO SHIP AND CA

	San Diego.		Los Angeles.		San Francisco.		
	Rate.	Amt.	Rate.	Amt.	Rate.	Amt.	
Pilotage (in) ..	$3 per ft. draft ...... 3 cents per ton	$270	$1 per ft. draft ...... 1 cent per ton	$90	$3 per ft. draft ...... 3 cents per ton	$270.00	$ 3
Water, survey and misc. (estimated).....	....	50	....	50	....	50.00	...
Dockage, per day	$2 plus 3/8 cent for each ton over 200....	285	For ship over 1 200 tons.. $14.75	177	Discharging: $4 plus 3/4c. for each ton over 200. Loading: 1/8 of above...	427.50	No
Stevedoring, per ton......	40 cents........	9 600	40 cents.........	9 600	40 cents.........	9 600.00	40
Handling, per ton	Included in "wharfage."	......	41.8 cents (av.)..	10 032	25 cents..........	6 000.00	
Dock rental, per linear ft. per month...	0	....	0	....	45 cents..........	225.00	..
Wharfage, per linear ft. per month	50 cents........	12 000	6½ cents (av.)...	1 560	5 cents (24 hrs.)..	1 200.00	50
Wharf storage, per linear ft. per month...	....	....	....	....	5 cents (48 hrs.)..	....	.
Pilotage (out).	Same as "in"...	270	Same as "in"...	90	Same as "in"....	270.00	Sa
	....	$22 475	....	$21 599	....	$18 042.50	..
Cost paid by ship..........	....	$875	....	$407	....	$1 242.00	.
Cost paid by cargo........	....	21 600	....	21 192	....	16 800.50	.

This table is the result of a comparative study of the rates in the different ports with a
to arrive at a condensed approximation of the cost of terminal charges and a comparison of
assumed of a 6 000 net ton register ship, 30-ft. draft, discharging and loading cargoes o
days. It is admitted that the assumption is not typical of ordinary occurrences, but it is ap
lack of uniformity in freight staples, it is well-nigh impossible to obtain averages excep
stevedoring has been taken at the uniform rate of 40 cents per short ton at all the ports.
equalize the cost by dictating the labor wage and the length of a day's work. Small n
lumped in the table at $50 per voyage. The charge for handling is a variable sum. The att
The dock rental, in San Francisco, for lines having regular assignment, is 45 cents per mon
per month. A monthly sailing for the case docketer will make the rental $225 per voyage.

AND CARGO IN PACIFIC COAST PORTS.

D.	PORTLAND.		PUGET SOUND.		VANCOUVER, B. C.	
Amt.	Rate.	Amt.	Rate.	Amt.	Rate.	Amt.
$270.00	$4.50 per ft. draft.... 3 cents per ton......	$315	$125 to $175......	$150	$1 per ft. draft...... 1 cent per ton........	$90
50.00	....	50	....	50	....	50
427.50	None.........	0	None...........	0	None...........	0
9 600.00	40 cents........	9 600	40 cents..........	9 600	40 cents..........	9 600
6 000.00	Included in "wharfage."	......	Included in "wharfage".	......	Included in "wharfage."	.....
225.00	....	....	0	....	0	....
1 200.00	50 cents........	12 000	50 cents (120 hrs.)	12 000	50 cents..........	12 000
....	....	....	25 cents per ton per mo. or frac. mo.	0	....	....
270.00	Same as "in".	315	Included in charge for "in" pilotage.........	0	Same as "in"	90
18 042.50	....	$22 280	....	$21 800	....	$21 830
$1 242.00	....	$ 680	....	$200	....	$230
16 800.50	....	21 600	....	21 600	....	21 600

ts with a view of determining the relative tax on commerce in each. In order
parison of the principal Pacific Coast ports, a purely hypothetic case has been
cargoes of 12 000 tons of miscellaneous freight, the ship remaining in port 12
but it is apparent that, because of the complex and diversified tariffs, as well as
ges except through an assumption that is somewhat strained. The cost of
the ports. This is not strictly true, but the labor unions may be relied on to
. Small miscellaneous items like water, etc., are insignificant, and have been
. The attempt has been made to arrive at an equitable average for each port.
per month per linear foot of dock rental. A 500-ft. space will amount to $225
oyage.

their very excess over any actual requirement; but nevertheless there will be an immense stride forward in definite development. How far it will extend in the near future will depend on the progress made before the tide of popular enthusiasm begins to ebb, as it may be expected to do within a few years after the opening of the Canal, and the shrewd promoters of port development and no less shrewd port engineering experts will doubtless make the most of the present opportunity and ride on the crest of the flood wave to a point which the normal depth of water would not permit them to reach.

San Francisco Bay.—San Francisco has a fund of $10 000 000 for port improvement, embracing sea-wall and dock construction and the acquisition of 64 blocks of submerged tide lands. A considerable portion of this sum will probably have to be used in replacing defective structures, but it is already decided to build twelve additional wharves of dimensions varying from 600 to 850 ft. in length by 140 to 200 ft. in width. Six additional ferry slips are to be provided, and the two sections of the belt railway are to be connected across Market Street. The approaching Exposition will doubtless stimulate to still greater exertion, particularly if San Francisco secures home rule in the management of her port.

Across the Bay, the City of Oakland has voted $2 503 000 in bonds and is actively at work in developing her west water-front and her important interior harbor. Her west front development, or the Key Route Basin, is a promising scheme, involving the filling in of 400 acres of land out to the bulkhead line between the Oakland and Key Route Moles, and wharfing out beyond this, still between the moles, to the pier-head line, in such a way as to form an interior basin with numerous slips and quays.

The plans for the interior harbor are scarcely less ambitious, while Alameda, Berkeley, and Richmond have their own projects of development. These have not yet taken form, however, in actually adopted plans or funds for carrying them out.

Los Angeles.—While Los Angeles has fought and is still fighting an heroic battle for the right to build and control a port of her own, she has as yet made only a small expenditure on her own account. She has, however, voted $3 000 000 in bonds and has committed herself to an expenditure of $7 000 000 more. She has had various studies made for the future development of the port, and E. P. Goodrich, M. Am.

Soc. C. E., has presented plans both for the outer and inner harbor which, if correctly reported by the press, are better fitted for a population like that of New York City than anything which even so thriving a city as Los Angeles has any right to expect in the near future. Fortunately, those dazzling schemes are harmless, and doubtless do good by broadening the public vision to a larger horizon.

The Goodrich studies provide an outer harbor in front of Terminal Island protected by a 20 000-ft. extension of the breakwater, and furnishing ten long piers, 500 ft. wide, with intervening slips 400 ft. wide. The inner harbor studies are more immediately practical, though scarcely less ambitious; but as they involve several alternative plans and have received no official action, it is scarcely worth while to discuss them here. It is enough to add that the actual realization of this plan for 82 miles of developed water-frontage will cost from $75 000 000 to $100 000 000.

According to the latest report of the U. S. Chief of Engineers, the City "is preparing to undertake immediately the improvement of 79 acres on the westerly side of the entrance channel * * * rendering available about 9 100 feet of deep-water frontage." This is in the outer harbor.

San Diego.—In San Diego no comprehensive scheme of development has yet been prepared, but the City has voted the sum of $1 000 000 for new docks and for sea-wall construction. The length of berthing space to be provided with this fund is 1 600 ft., and the length of the sea wall is to be 2 500 ft. This will make possible the reclamation of 50 acres of tide flats. No definite steps have as yet been adopted for a terminal belt railway. It is stated that the City intends to follow up its present plans with a $3 000 000 bond issue. Perhaps most important, as bearing on the future of the port, is the project, said to be in contemplation, of building a railroad directly east to Yuma.

Portland.—Portland, like San Francisco, is pursuing the same course of "procuring her cloth before cutting out her garments." In 1910 a Commission of Public Docks was created by City Charter amendment, and the sum of $2 500 000 was voted for the construction of such docks as might be needed in the near future. This Commission employed a board of experts from New York City, consisting of E. P. Goodrich, M. Am. Soc. C. E.; W. J. Barney, Jun. Am. Soc. C. E., and Charles W. Staniford, M. Am. Soc. C. E. A complete de-

tailed plan has been prepared by the Board and has just been made public. Those features of the plan which are proposed for immediate adoption consist of two combination docks, sheds, and warehouses, one on each side of the river, a coal dock, a motor-boat landing, fireboat station, etc.

The West Side improvement will consist of a quay dock, 1 075 ft. long and 100 ft. wide, in the central section of the harbor below the city bridges. For two-thirds of its length it will be a single-level structure, and the rest two-level, the upper being at Elevation 32 and the lower at 18 ft. above low water. The dock will be provided with adjustable slips, escalators, and elevators, and will have space for storing 10 000 tons of freight. A six-story reinforced concrete warehouse provided with full equipment for handling goods will be erected in the rear of the dock. Its capacity will be about 40 000 tons. Three additional warehouses are planned as soon as funds are available. Dock autos will be the main reliance for handling freight.

The East Side improvement, conveniently located for the transaction of business, consists of a two-story quay structure, 500 ft. long by 100 ft. wide, capable of storing 6 000 tons on each level. A one-story reinforced concrete warehouse will be provided with a foundation for five additional stories. All structures in both improvements are to be of steel and concrete, and fire-proof throughout.

In addition to these improvements for immediate adoption, the plan of the Consulting Board provides for future development on the two sides of the river, 7 000 ft. of quay construction, pier and slip development affording 21 200 ft. of berthing space with piers 300 ft. wide, together with the necessary warehouses and other accessories. A large coal-handling plant, freight-assemblage and storage yard with a capacity of 1 500 cars, and a public belt line serving all the docks, are also to be provided. The total development gives more than 6 miles of berthing space, of which only about 1 750 ft. are as yet provided for. As to this initial programme, it is understood that it is acceptable to the Dock Commission, and that steps will be taken immediately to carry it into effect. In connection with the Dock Commission projects, reference may again be made to the joint projects of the Government and the Port of Portland Commission to secure a 30-ft. channel to the sea.

Puget Sound and Grays Harbor.—North of the Columbia River, port-development work will doubtless always be associated with the name of Virgil G. Bogue, M. Am. Soc. C. E. Pursuant to a charter amendment of the City of Seattle adopted March 8th, 1910, a Civic Plans Commission was created for the purpose of forming a plan for the future development of that city. This Commission employed Mr. Bogue as its Engineer, and his very elaborate report, submitted August 24th, 1911, includes a complete scheme of development of Seattle's harbor. Following immediately on the submission of this report came the creation of the Port Commission of Seattle, specifically charged with the duty, among others, of preparing a "comprehensive scheme" of port development. While Mr. Bogue's plans as a whole have not been adopted by a vote of the people, their essential harbor features will be given close study in the preparation of any plans that the Port Commission may adopt. Mr. Bogue, after the completion of his Seattle work, made a similar study for Tacoma and also for Grays Harbor, and he is now engaged on a plan for Prince Rupert. As already stated, whatever action may be taken as to the immediate adoption of his work in these ports, it is certain to have a profound influence on their future development.

The Seattle plan embraces elaborate suggestions for every part of the harbor, with especial emphasis laid on the Harbor Island and Smith's Cove sections. The mosquito fleet and industrial features of the inner harbor are likewise fully elaborated. No effort was made to form an estimate of costs, for the plan is one for long-continued development; but it certainly represents an outlay of not less than $25 000 000.

The Port Commission has definitely adopted an initial programme involving an expenditure of $6 000 000. This includes a slip with two wharves on the east side of East Waterway, giving berthing space for four large ships; a solid filled pier on the central water-front, with a 1 100-ft. berthing space of deep-sea depth, and a floating landing stage for small boats with 800 ft. of landing space, and also a large fruit cold-storage plant; a lumber, coal, grain, and general transhipment dock at Smith's Cove; a small dock for general use on Salmon Bay; and a new ferry across Lake Washington. The Port Commission is taking initial steps looking to the establishment of a terminal railway system for the port.

In addition to the Commission's original programme there is being considered, at the earnest request of certain business interests, a very ambitious project for Harbor Island patterned after the Bush Terminals of New York. The pier development proposed comprises six new piers with an aggregate berthing space of 16 800 lin. ft., or enough for the simultaneous berthing of 42 ships of an average length of 400 ft. Back of the piers is to be an extensive industrial development. The whole project is to be a co-operative one between the Port District and a private terminal company.

It is conservative to state that expenditures of all classes—Federal, State, County, City, Port District, and private—actually authorized or in definite contemplation, for the development of Seattle Harbor, aggregate approximately $20 000 000.

The study made by Mr. Bogue of the possibilities of Tacoma Harbor may best be told in his own words as summarized in his report on Grays Harbor:

"At Tacoma, a great movement is on foot to keep the city well to the fore in all port extension matters. The plans contemplate a canal system with an aggregate length of 6 miles, to say nothing of spurs and laterals which may be built in future years in connection with large industrial enterprises. The cost of this canal system is estimated at about $4 000 000. It will develop a vast area of tideflats and present unexcelled opportunities for industrial development with convenient access from both sea and land. Beside the canal system a number of waterways are to be built and various slips and piers which will provide the best accommodations for vessels engaged in ordinary commerce. When these improvements shall have been made, the wharf frontage of Tacoma between Point Defiance and Brown's Point will aggregate 42.6 miles, all of which is well protected and easy of ingress or egress."

Olympia, Everett, and Bellingham are not as yet taking positive action in the development of their ports, but they will probably do so in the near future.

Grays Harbor.—The Port Commission of Grays Harbor, through Mr. Bogue, has prepared an elaborate scheme of development, but the report does not furnish an estimate of its probable cost, and no provision is yet made for carrying it into effect.

Vancouver, B. C.—As already stated, no comprehensive scheme of development of this port has been adopted. Special studies have been

and are being made of different parts of the harbor, and there are rumors of gigantic development plans by private interests. The Great Northern Railway has a fine new pier under construction and other private improvements are in progress; but there is as yet no general plan, no distinctive scheme of port administration, and no provision of public funds.

The conformation of the harbor and the handicap of tidal fluctuation and currents have suggested an improvement which, though costly, would seem to possess great merit. That is to place a lock and dam across the throat of the harbor at the second Narrows, thus holding the upper part of the Inlet at high tide and making it a fixed-level fresh-water harbor. This would restrict the tidal area to one-third its present extent and would reduce tidal currents in the lower harbor so much as greatly to improve conditions there. It would seem on the face of it that this would be an improvement of great importance to the future of the port.

New Westminster.—The fresh-water port of New Westminster is taking active steps in harbor development. It has recently expended $15 000 on the study of a plan and its presentation and is about to put into effect the one prepared for it by A. O. Powell, M. Am. Soc. C. E. It comprises wide marginal streets, routes for railways, the extension of the city quay, and a long waterway in a slough parallel to the river, from which the river is to be excluded at the upper end. A succession of parallel slips, oblique to the axis of the waterway, will develop a berthing space of 7½ miles.

Victoria.—Even that sometime staid and delightful provincial town of Victoria, more distinctly British than any other port of the Pacific, has become infected with the Panama *bacillus* and is cogitating what it may do to be prepared for the changes which it has been told are about to ensue. No definite plans have as yet been evolved, but among the tentative suggestions are an outer harbor to be protected by a breakwater in front of the present entrance to the inner harbor; and also a development in Oak Bay on the opposite side of town fronting to the eastward.

Prince Rupert.—Studies for the Port of Prince Rupert are now in progress under the direction of Mr. Bogue, but are not yet far enough along for incorporation in this paper. A suggestion that has been made of possible future development in this port, however, may be

properly mentioned. Prince Rupert City is on an elliptically shaped island about 6 miles long north to south by 4 miles east and west. The "lakes" which separate the island from the mainland in the rear have an area north of the railroad crossing of about 40 sq. miles. It is not unlikely that the north and south passages will be dammed off and provided with locks, thus converting the lakes into a tideless, fresh-water harbor just at the level of high tide. While this development is still some distance off, it has such great advantages, in view of the extreme tidal fluctuation in Prince Rupert Harbor, that it will probably be realized eventually.

Outlay on Pacific Coast Ports.—From the statistics of obligations actually assumed to date, and by the aid of the most reliable forecasts obtainable of port planning for the near future, it will be a conservative estimate to place the cost of port development along the Coast during the next five years at $50 000 000.

VI.—Influence of the Panama Canal.

The writer will close with a brief statement of the influence of the Panama Canal on the commerce of the Pacific Coast. The aggregate effect will be a summation of the following:

(1) Effect on consumption and production among the existing population.

(2) Effect on consumption and production through increase of population—chiefly immigration.

(3) Effect on rail commerce between the Pacific and Atlantic Coasts.

(4) Effect on industrial development.

(5) Effect on Oriental traffic to the east coast of America now transhipped to rail at Pacific Coast Ports.

(6) Adverse effect of Canal tolls.

Consumption and Production.—Under the first heading, it may be assumed that there will be some increase of consumption due to a reduction of prices resulting from lower freight rates *via* the Canal. For example, anthracite coal may possibly find considerable use along the northern coast where now it finds very little, if any at all. As to production, the effect ought to be extensive and immediate on lumber products, for the Canal will open to Coast lumber the market of both shores of the Atlantic, and the production can be instantly increased

to any desired extent by putting existing mills now closed into operation or speeding up others to their full capacity. In the matter of fruit and cereal products, no radical effect on production should be expected as an immediate result of opening the Canal. The conditions are not the same as with lumber, and any material increase of production must await an increase of population and general development.

Immigration.—This consideration brings up the second heading specified—the influence of the Canal on the increase of population on the Pacific Coast. It is an influence which, in the long run, may prove the most important of all; for it is not so much additional port facilities that are needed to develop a country as increased numbers of those who do the work of development. Many other factors, of course, contribute; but in a new country increase of population is the main thing, and this must come in large part through accessions from the outside. There is a general consensus of opinion that the opening of the Canal will have more immediate effect on immigration than on any other class of traffic. In this fact lies a great hope and a great peril for the Coast. What kind of immigrants will be brought here? What reception will they meet? Will they prove a blessing or a curse to our communities? The writer will consider these matters no further than to quote a paragraph from a bulletin recently published by the Seattle Port Commission:

"In this connection the commission may be justified in digressing so far as to observe that the real preparation for this influx of immigration rests with the people of the State rather than with the Port Commission or the Government. If intelligent and organized effort is made through agents in Europe to secure a good quality of immigrants, and if similar effort is made here to start them in their new career without too much hindrance and discouragement and without being victimized by those whose only desire is to fleece them of their meager belongings and leave them stranded in a country where the problems of successful agriculture are so different from what they are accustomed to, this immigration may be of inestimable value to the State and indirectly to its metropolis and chief port. But if this matter is allowed to drift along without organization or sympathetic effort in aid of the new arrival, the result may be to crowd this port with an indigent population and to justify the effort which is said to be already under way here to resist the whole immigration movement."

Diversion of Rail Traffic.—In reference to the third consideration, a degree of uncertainty exists which completely nonplusses the ablest

traffic authorities of the country. The rail traffic between the Pacific Coast and points far enough east to be affected materially by the Canal is smaller than is popularly supposed. A high railroad authority has estimated that not more than 600 000 tons of traffic goes to the Northwest Coast from east of "Missouri River common points", and that only about 20% of this originates east of Buffalo or Pittsburgh where the influence of the Canal would be effective. On traffic from the Great Lakes and from the Mississippi, Ohio, and Missouri Basins, it is not likely that the Canal will have much effect. The railroads will hold it except for the possibility that it may cease to originate in those sections. All but about 7½% of this traffic, it has been estimated, might originate on the Atlantic and Gulf Coasts, where it would fall directly within the Canal influence. Is it not likely that the ultimate result of the Canal will be to transfer to the Atlantic Coast much of the Pacific Coast traffic which now originates in the central sections, thus cutting off from both the railroads and the producer of the Central West an important business? And is it not a perception of this probable result which accounts for the lukewarmness, if not active opposition, of certain sections of the country to free tolls for coastwise trade through the Canal? In any event, it seems to be reasonably certain that the Canal (particularly with free tolls) will transfer from rail to ship a large part of the traffic which now crosses the country to the westward by rail. This will operate to increase the business of ports on both coasts and of the railroads from the coast inland for uncertain distances.

As to east-bound traffic originating on the Coast, lumber shipments will increase very largely, but rather through the opening of new markets than the diversion of existing rail shipments. The present eastern market for Coast lumber is mainly in the tramontane region eastward as far as the Mississippi. The market farther east cannot stand the heavy freight charge except for shingles and the finer grades of lumber. The market of the Central West will continue to get its supply by rail, and only consignments farther east are likely to be diverted to the water route.

With fruit products of all kinds along the Coast, there is every reason to believe that the Canal will effect a marked change in the method of shipment. Refrigerator ships will carry this product to both shores of the Atlantic in better condition and at cheaper rates

than can be done under present conditions. It is estimated that within 10 years the fruit shipments from Central Washington and Oregon will approach 100 000 carloads, while the shipment of all classes of fruit from California will exceed that figure. The problem of handling those prodigious quantities by rail in the short periods and over the long distances necessary may prove an impossible one; and the water route will come in as a great relief in addition to its normal advantages. It seems one of the certainties of the near future that cold storage plants of immense capacity will be built in all the principal ports in order that the fruit crop can be stored in part, awaiting shipment.

In regard to wheat and other grain and fish products, the Canal will not make much change, because these products already go largely by water; but considerable change in the method of water shipment is likely to result. Wheat, for example, is now shipped mainly in sacks in order to avoid the alleged danger of heating if shipped in bulk on the long voyages *via* the Horn, Good Hope, or Suez. The shorter voyage *via* Panama will obviate this difficulty, and we may expect to see elevators in greater numbers provided in our ports, as they are on the Great Lakes, at Montreal, and in other Eastern ports.

Industrial Development.—It is a much disputed question as to what the effect the Canal will have in promoting industrial development on the Coast, particularly manufactures. Some hold that the lower freight cost will bring raw materials here at rates which will make manufacturing practicable where it is not at present. On the other hand, it is argued that the lower rates will bring manufactured products from the East more cheaply than at present, and that this will offset the tendency just noted. Where the balance of these conflicting forces will fall is wholly problematical, but it is certain, from the uniform experience of the older sections of the country, that increasing growth in population and wealth of itself will bring increased development of manufactures. It will become a part of the development of every port and will operate to increase its business.

Oriental Traffic.—As to Oriental traffic transhipped to or from Atlantic or Gulf points, it is clear that the effect of the Canal must be to draw it away from Pacific Coast ports. It will not draw it away altogether, because with certain commodities, like silk, of high value in proportion to freight charge, the interest on purchase cost may amount to more, through the longer time required to receive the goods *via* the

Canal, than the saving in freight. Most of the manufacturers consulted are agreed that silk shipments will continue to go by rail. With other articles, however, such as hemp bound East and cotton bound West, it would seem that, if through lines are established direct to the Orient, *via* Panama, they must absorb this class of traffic.

Canal Tolls.—It is a matter of profound satisfaction to be able to state in this discussion of the ports of the Pacific Coast that a question vital to their welfare and one but yesterday of doubtful issue has been settled and settled right. Our coastwise trade will pass through the Canal without paying any toll. The great principle of the free public highway, and of the free use of public improvements of rivers and harbors, has triumphed over the narrow vision and selfish interest that threatened to defeat it. It is one of the greatest victories of an enlightened national citizenship, which illumines the record of this generation.

The victory, however, great as it is, is not as great as it should be. The Canal should be free to all commerce. The highway toll system on this great thoroughfare should be abolished for all nations and forever. The same system on the public roads became its own executioner, because it defeated its own end. It prevented the use for which the road was built, and while it was an improvement on no road at all, it never performed its full service. So likewise, tolls on commerce through the Panama Canal will restrict the rightful use of that great work. This is a demonstrable certainty. Take a table of distances and look up, for example, the routes from New York to Valparaiso *via* Panama and Magellan. The Panama route is 3 500 nautical miles shorter. Yet it is a fact that the delay in passing the Canal plus the toll cost at a dollar a ton commúted into cost of running the vessel will make it cheaper for large fast vessels to take the longer route. To vessels between Valparaiso and European ports, the disparity against the Canal route will be still greater, and, of course, the same principle applies in varying degrees to all routes. Tolls will inevitably restrict the use of the Canal, and thus deprive the United States and the rest of the world of its full benefit.

It is one of the most powerful illusions that public thought is subject to, that a work like this, in order to justify itself, must be made to pay its own cost in direct revenue. It is easy to overlook the fact that every dollar collected in this way has to be paid by the people in

some other form. At the very best it is taking out of one pocket to put into another, and meanwhile the utility of the work is being curtailed by restriction of use. Happily the true principle has been grasped by our legislators and applied to the Canal as far as our own trade is concerned, and that is the utmost which it was in their present power to do. It is only just that foreign nations which have borne no portion of the cost of the Canal should pay something for its use. No one can question the correctness of that position, but this end should be attained by payment for this use without imposing a toll by the discovery of some method which will not restrict the use of the Canal. Is such an achievement possible? The writer believes that it is.

It can come, of course, only through international agreement; but it is conceivably practical to form a convention by which each nation shall pay into the Treasury of the United States a proportion of the rightful charge based on the actual use of the Canal by its own subjects, or under its own flag, or on the tonnage to and from its own ports. The charge would be computed on cost of maintenance and operation and such an additional sum annually as would retire the indebtedness within a determinate period. The question is surely worthy of highest consideration, and few subjects more vital to the interests of the world at large could engage the attention of the Third Hague Conference in 1915 than this of mutual co-operation to make the greatest public work of the ages free to all users. Let the police jurisdiction, direct control and operation, and questions of use in time of war be left to the United States; but otherwise let it be free. As the writer has elsewhere said, a "way must be found for making Panama and Suez as free to the world's commerce as if Nature herself had excavated their channels—as free as are the Straits of Gibraltar and Magellan."

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

THE APPRAISAL OF PUBLIC SERVICE PROPERTIES AS A BASIS FOR THE REGULATION OF RATES.

Discussion.*

By C. E. Grunsky, M. Am. Soc. C. E.†

C. E. Grunsky, M. Am. Soc. C. E. (by letter).—The point which the writer particularly desired to emphasize in the paper was that when rates are to be fixed the investor in public service properties is entitled to adequate protection. The method of securing this protection intelligently was described, and the writer tried to make clear that there should always be an allowance in some form and under some name to compensate the owner of the public service property for hazard, and for management both during construction and under operation. This compensation should appear in the rate of interest to be earned rather than in the appraisal on which interest is to be allowed. Placing it in the appraisal is a convenient way of making the earnings, expressed in percentage thereof, appear low. In whatever way this may be handled, something more than ordinary interest on the properly invested capital should appear in the earnings, if these have been equitably fixed. Mr. Grunsky.

There is no intent to advocate the taking of any property without due compensation, as Mr. Higgins seems to infer from parts of the paper. When the broad principle is laid down therein that the owner of a public service property is entitled to be protected to the extent of his investment, first, in the matter of receiving thereon a proper return, and, second, in having the invested capital itself protected

* Continued from August, 1912, *Proceedings*.

† Author's closure.

Mr. Grunsky. and ultimately returned, this is not to be understood in a strictly restrictive sense. There may be elements not covered by actual investment, which are, nevertheless, essential parts of the property and represent investment even though never actually paid for by the owner, and not appearing as having cost him anywhere near their market value. In such cases, however, it will generally be well to inquire into the circumstances attending their acquisition.

Water rights, such as Mr. Van Cleve cites, are to be considered as property which has value. As part of a public service property, they may or may not represent actual investment. Such water rights, nevertheless, are to be classed as elements to be included in the appraisal. Probably no hard-and-fast rule can be laid down for determining their value in the sense of what it might reasonably be assumed that they would cost if they had to be acquired from other owners. The circumstances attending the valuation, particularly of undeveloped water rights, are so varied, involving as they do all the uncertainties of present and future demand for power, that any satisfactory and conclusive suggestion along this line is hardly to be hoped for at this time.

In California the flowing water belongs to the people, but the riparian owner on a stream has certain paramount rights. A distinction is to be made, therefore, between a water right in the broad sense in which it is here generally referred to, when it is strictly a grant by the people on a par with a franchise, and, in the more restricted sense, comparable with the illustration used by Mr. Van Cleve, in which case the power right is the property of the riparian owner. In the one case the water right deserves the same treatment as a franchise; in the other case the fact that it has a market value is easily recognized, and its appraisal is a comparatively simple matter.

In reply to the question which Mr. Boucher asks concerning the appraisal of property such as a valuable right of way which cost little or nothing, the circumstances in each case should be taken into account. In the case which he cites, the railroad is not entitled, in strict equity, to a return on the full amount that the property would cost to-day, but only on a fair allowance for investment. In other words, there should be some limit to the right to a return on the unearned increment represented by the present high value of the right of way, if such value be measured by what it would cost if it had to be acquired to-day. This view appears to be in accord with the rulings of the Courts, which would give to the public service concern, practically as a bonus, the appreciation in the value of such properties. What shall be considered excessive appreciation, is a question which had best be answered only as special cases arise.

A similar question is that relating to the treatment of donations. Take, for example, a water company which is called on to extend its

service into new territory not yet built on, the owners of which construct a distributing system of pipes and house connections and all that goes toward a satisfactory service and donate the same without cost to the water company. To add the cost of such property at once to the capital invested by the water company might work hardship and be unfair to the older consumers. When the new territory is developed, and the consumers, resident therein, take a fair share of the water, it may well be asked whether or not in fixing rates the fact should be taken into account that a part of the system was donated by the water users themselves. It would be equitable to deduct donations from the appraisal, allowing, however, adequately for the upkeep of the entire system, including depreciation. On the other hand, there is good basis for the view that it makes no difference how the property is acquired, and that the appraisers for rate-fixing purposes should ignore the fact that some of it may have cost nothing. Mr. Grunsky.

The difficulty that confronts the rate-fixing body in matters of this kind arises from the generally accepted theory of the past—as advanced by owners of public service properties—that they are entitled, as profit, to whatever they can get in the shape of bonus and also to the full amount of the unearned increment, represented by increased value of the elements which go to make up the whole of the property. In order to avoid controversy in the future, these matters should be made clear, and it is by the general recognition of fundamental principles that this can be brought about and misunderstandings avoided.

The question is asked by Mr. Coombs as to whether high dividends paid in the past should be taken into account in making appraisals. In a general way, yes. Consideration may be given to this fact just as well as to the fact that there have been lean years. Ordinarily high dividends would hardly be construed as repayment of invested capital, but when such dividends have been paid to the detriment of the property, when proper foresight has not been exercised in making provision for its expansion, when the requirement for deferred maintenance is high, it may be that such conditions are due in part to the high dividends that have been paid, and this is then a circumstance to be considered.

The method of dealing with depreciation, as indicated by Mr. Kiersted, is noted in the paper as applicable to any public service property in which the depreciation may fairly be considered equivalent to the average annual replacement requirement. This is set forth in Paragraph 14 of the fundamental principles and elsewhere. The bookkeeping in such a case would be as Mr. Kiersted describes it.

Such elements of value as those enumerated by Mr. Kiersted under his definition of "going concern" may properly be included in a valuation, and perhaps the designation, "going concern," is

Mr. Grunsky.

properly applied to them. Against their inclusion no protest is raised. The protest which is implied in the paper relates to the addition of intangible values, whether under this or other designations, to a demonstrable investment, or, as is more frequently the case, to an estimated value less depreciation, in order to bring the same up to what the appraiser thinks it ought to be. If there is to be any addition to the investment appraisal, let this be clearly stated, and it will make no difference under what designation it is added; or, as pointed out in the paper, let the appraisal be made without deduction of depreciation, and let the percentage rate of earnings be sufficiently high to do full justice to the owner of the property.

In closing, the writer wishes to express his appreciation of the reception which his paper has received and to thank those who have contributed to the discussion for emphasizing and making more clear some of the thoughts which were but imperfectly presented.

In this connection attention may be called to Mr. Kiersted's amplification of the writer's brief comment on the perpetual life of composite public service properties. This idea was incorporated by the writer, in 1901, in a report on the appraisal of the properties of the Spring Valley Water Company, which is the company supplying San Francisco with water. If properly applied the assumption of perpetual life will be found helpful in many cases where, for a plant taken as a whole, no fixed or definite term of life can be assumed.

In the case of a complex plant, such as was then under consideration, it was reasonable to fix its probable life at so long a period that it might be called perpetual. Parts of it would deteriorate and go out of use; parts of it, except for the highly improbable case of its being killed by a competing system, would continue in service practically for all time. As a whole, therefore, and for rate-fixing purposes, it could be considered as being always in good condition. There was no need of writing off depreciation. The replacement of such parts as went out of use could be cared for under maintenance and operation.

This idea naturally resulted in the adoption of a method of calculation described in the paper as the method under which no deduction need be made for depreciation in valuing public service properties for rate-fixing purposes.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

ENGINEERING EDUCATION IN ITS RELATION TO TRAINING FOR ENGINEERING WORK.

Discussion.*

By Messrs. N. B. Garver, George B. Pillsbury, F. H. Constant, and Arthur B. Green.

N. B. Garver, Assoc. M. Am. Soc. C. E. (by letter).—This paper has been read with much interest. Many of the statements made therein the writer believes to be true, but there are some things which cannot be accepted without a grain of salt. For example, the author advocates a school year of four terms of 12 weeks each, the student being enrolled in any subject of his course in any one of the four terms of the school year. This means that every subject required in a given course must be taught during every term, a condition which is impossible to realize in practically all engineering colleges, unless the instructional staff is doubled or trebled, and unless the number of classrooms at the disposal of the teaching force is materially increased. Few of the engineering colleges have the financial backing to permit this. Mr. Garver.

Again, the author advocates an idea which is not considered practicable in engineering construction work: He thinks the practicing engineer should specify the course of instruction and the way it should be imparted, and that the builder—the school—should guarantee the quality of the product. To quote:

"He is a stronger man who adds to his natural ability the power given him by the pursuit of a carefully arranged course of study in the essentials of engineering science at a well-conducted school. This being the case, it should be settled by engineers, and not teachers, just what instruction is best and how it should be imparted."

*This discussion (of the paper by Ernest McCullough, M. Am. Soc. C. E., published in *Proceedings* for May, 1912, and presented at the meeting of September 4th, 1912), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Garver.

Again:

"Manufacturers of materials used by engineers have been compelled by competition to furnish products satisfactory to the user, so why should not the schools, that manufacture embryo engineers?"

The writer agrees with the author that the more nearly the atmosphere of the college of engineering approaches that of the office, and not that of the school, the better it will be for the student. The difficulty with a great amount of the work done by the student is that he does not see any direct connection between the work he is doing and the practice of engineering. Much of his work could be arranged so that it would be done in practically the same way as office work. Too many students acquire the habit of doing things in a happy-go-lucky sort of way, working when they feel like it, or spending their time in social functions, rather than in some work that will advance them in their chosen line. Most students should be placed in such a position that it would be necessary for them to develop a systematic way of using their time and also a systematic way of doing things. It has been observed by the writer, on more than one occasion, that the student whose time is fully occupied, and who must necessarily use system, does work of a better grade than one whose time is not fully occupied.

The young man who completes an engineering course in a college is not an engineer, and employers should not expect such in engaging technical graduates. The writer does not believe it is within the province of the engineering college to teach men to be expert draftsmen or what-not. A good letterer is produced by practice and not by college training. The practicing can be done as well outside of college as in it. The engineering college should give to the student, during the four years he is in attendance, the things he cannot gain through his own efforts. The facility with which a student may get a job and hold it is not the only thing to be kept in mind in engineering education.

Mr. Pillsbury.

George B. Pillsbury, Assoc. M. Am. Soc. C. E. (by letter).—The remarks and recommendations of the author seem to indicate an opinion that the technical schools should advance yet farther in their tendency toward practical training. The writer must confess to the view that both the technical and the secondary schools have already advanced too far in this direction.

What, indeed, is the object of the technical schools? The author regards it as the training of young men to be proper engineering assistants, who may develop, if they have the opportunity, into successful engineers. The writer would suggest that the object should be rather to prepare the students, so that in due time, if they are able, they may become successful engineers.

If the object of the technical schools were the training of assistants, the writer would be inclined to join those who decry the schools and all their works. For who could recommend the expenditure of time

and money necessary to complete a course at a technical school, if the end were the preparation of the student as draftsman or instrument man? The concentration and discipline of an office or a field party will teach these duties in far less time and at far less cost; nor can the schools hope to turn out men whose skill and knowledge of details is sufficient to make them immediately proper engineering assistants. The profession is of far too wide a scope, and is divided into far too many branches, to make such an ideal possible. Mr. Pillsbury.

It would appear then, that the function of the school should be the teaching of those things which are not taught in the field and office. They should teach the fundamental "whys" of the profession, rather than afford a smattering of the infinite "hows." More than this, they should strive to discipline the minds of the students into that clear thinking which leads a man to view his task in all its bearings, so that it may be done wisely, in conformity with the spirit of his instructions, and not mechanically, in conformity with the words.

It will be generally admitted that, as a means of mental discipline, nothing can equal a rigorous course in pure mathematics. A "practical" course only encourages the habit of loose thinking; and, if the mind be properly trained, the practical applications are easily acquired. The course should not be so long, it is needless to say, as to interfere with the sciences, and with the theory of the mechanics and the materials of engineering, but it should be rigorous, and not "practical."

No disparagement of the so-called culture courses is intended. These studies, if really taught, not administered as sugar-coated pills, must give the student a broader view of life, to his material advantage.

A real knowledge of written French and German opens sources of information which are not to be despised.

Practical instruction must be given, to maintain the interest of the students; but as such instruction, at a school, is essentially impractical, it would appear that it should be definitely regarded as a means and not the end. It is suggested that the need of the schools is more concentrated effort on the part of the students, rather than a reduction of the academic side of the curriculum.

F. H. Constant, M. Am. Soc. C. E. (by letter).—Every engineering instructor who reads Mr. McCullough's paper will appreciate the justness of many of his criticisms, and will be deeply interested in his suggestions for the improvement of the product of our engineering schools. That this product is not wholly what it should be, is a statement which need not be debated. It is of more immediate concern to learn wherein lies the explanation, and how the conditions may be remedied. Mr. Constant.

The technical schools are confronted with the following conditions: The grist that comes to their mill is of a more varied kind and

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in more overwhelming quantity than ever before. Every parent, whether well-to-do or not, seeks to send his children to college, and in these days of wide-spread and cheap educational opportunities, this is not difficult to do. There is a general turning away from all manual and artisan occupations toward professional and business careers. Engineering, which formerly was a little known and somewhat contemptuously regarded vocation by those outside its ranks, has leaped into a well-deserved popularity. It is now regarded as a fine, manly and, withal, genteel profession, full of interesting possibilities, appealing to the constructive instincts of a man, and developing his best mental and moral forces. There is also a lurking impression that it is a lucrative calling.

The result is the hundreds of young men who knock annually at the doors of the engineering colleges. Many of these have little or no conception of the nature of the profession they seek to enter; some are by nature unsuited; and not a few are there only at the earnest solicitation of parents who know little about engineering but think it is a fine profession for their son. Nearly all instructors lament the fact that the average present-day student lacks the earnestness, singleness of purpose, and interest in his work that characterized the smaller classes of a few years ago, and this is probably true. Formerly, few entered an engineering college who did not know what they were there for and how to get it. Engineering was not then so complex; the need was for men trained and expert in technical details; and this kind of training the college is eminently qualified to give.

The trouble is that the engineering college has not fully recognized the changed conditions, and has sought to patch up the old machine to meet present-day needs. Instead, a very different raw material comes to its hopper; the internal treatment needs to be modified correspondingly; the mechanism for eliminating the unsuitable requires to be made more effective; while the final product itself should be of a somewhat different character than formerly. This is the situation which confronts the engineering college of to-day.

The first and most natural effort of the engineering college is to try to reproduce the old conditions, with which it is already familiar, by raising the bars of admission. That there should be a radical reassortment of the material which seeks to enter the Engineering Profession is apparent to all, both inside and outside of the college. The cogs of the most perfect machine cannot run smoothly when it is called upon to digest too large a proportion of unfit material. The country, however, needs—much more than it needs more young engineers—men who are skilled in the use of their hands as well as minds, trained artisans and handcraftsmen. It is time to rid ourselves of the self-satisfactory delusion that the American workman is the most clever craftsman in the world. This may be true along some narrow

lines, but, in general, we are unintelligent and inefficient in the combined use of hand and brain. One has but to pick up any article or device requiring a little intelligent craftsmanship to find, stamped somewhere upon it, "Made in Germany." Why should it be necessary to import from some other country everything requiring taste, thought, and manual skill in its production? We are not the most clever people, but, at the same time, we are not the most stupid. Our ranks are annually recruited from the artisans of these very countries which thus outstrip us. We need more schools, and many of them, for the training of skilled artisans. We seem, in our haste to get the material goods of this world without working for them, to be developing a nation of business men at the sacrifice of trained workers. The result is restlessness, discontent among all classes, a depreciation of the skilled trades, and the high cost of living. The establishment of the higher trade or craftsmanship schools would deflect away many who now enter the engineering colleges, to the mutual advantage of both. Mr. Constant.

Probably nearly every instructor, who theoretically favors lowering the bars of admission, rather hesitates to see it put into operation, through an instinctive distrust of the efficiency of the processes of elimination within the college. The judicial side of the instructor's work is the most difficult and distasteful, requiring acumen, experience, moral courage, and a true knowledge of correct standards. He is assailed, on the one hand, by every pressure which the weak or lazy student can bring to bear, while, on the other, the judgment of the profession at large is indefinite and somewhat remote, and is only felt after cumulative years of experience with the product of the college. The line of least resistance (and often of inclination) is to yield to the student. That such a large number of unfit men is actually eliminated from college, indicates that the average instructor is conscientiously "on his job." That a further improvement is needed, is apparent to all. With a greater perfection in the mechanism for ejection, the entrance bars may well be lowered, as suggested by the author.

After his entrance, the first work of the college is to care for the student and properly prepare him for his life's work. The author has suggested a curriculum which will probably not elicit a great amount of criticism. The details and arrangement of the curriculum may, perhaps, be confidently left to the colleges to develop. Most of the technical professors in the leading engineering colleges have been and are practicing engineers, while the younger men teaching the same subjects are generally drawn directly from the ranks of the profession. It may be assumed, therefore, that the engineering college is in close enough touch with the active profession to understand the nature and quality of the work it should perform. Nevertheless, every member of

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the teaching profession will welcome constructive suggestions like those presented by the author.

After all, however, it is not so much the precise nature of the curriculum as the manner in which the subjects and the students are handled that is important. How to bring out the very best in every man, to stimulate his interest and devotion to his work, and, at the same time, to eliminate the lifeless and the small group of deficients always to be found at the lower limit, who, by sheer persistence, in point of time, finally get through, no more fit, perhaps, at the end than at the beginning—this is the real problem of the engineering school.

The author has suggested that the office atmosphere should early be introduced into the classroom, and with this the writer is in full agreement. It is somewhat incongruous, to say the least, that a young man of about the age of twenty should be handled by the same academic methods as those used in the high school, and that he should be led to believe that success is measured by the attainment of a certain grade mark, whether he really knows anything about the subject or not, or by the mere making of certain design drawings or laboratory experiments, whether accurate or not, or whether largely inspired by others or not, when immediately he steps out of the college door into the real world of his calling, for which he is supposed to be fitted, he finds that he is judged by entirely different standards—not by what he pretends to know, but by what he actually knows; not by what he knows, but by what he can efficiently use; not merely by what he knows, but by what he does not know, but knows where and how to get it; not by what he can use 70% accurately and 30% inaccurately, but by that of which he has a command of 100%—in a word, by his accuracy, real efficiency, self-reliance, and vitality as a growing and expanding force. If the college cannot even suggest to him what is in store for him when he leaves its walls, it has failed to live up to the measure of its opportunity.

Probably there is little room for change in teaching methods in the earlier subjects of the curriculum, such as mathematics, physics, etc., except that even these should always be taught with the future needs of the engineer in view. A good deal of fine judgment in the elimination of the unfit should be exercised. A large part of the weeding out of the unfit must be done in these earlier years, both in justice to these men and to keep the wheels of the machine from becoming clogged. Men should be judged, not solely by their marks, but by their real proficiency in each subject; and the unfit and lazy should be ruthlessly and firmly ejected. Even the professor of mathematics should constantly seek to discover those qualities which promise to make future good engineers. For the product of the engineering college is not mathematicians but engineers. The professor's work,

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from the beginning, should smack of office and field, and accuracy and efficiency should be striven after, rather than a great range of knowledge.

In the higher technical work, especially that of designing, the office atmosphere may be exactly reproduced, and the men put upon their mettle and judged by the same standards which they will meet when they leave college. In this way there will be no abrupt transition between the life of the college and the working world; and this, if the writer is not mistaken, is what the author and all other employers of young engineers desire. The creation of the office atmosphere and the reproduction of professional conditions is the problem which the instructor must solve, and which, to a large extent, depends on his own personality and experience. In engineering, as in every other department of learning, the college is not the walls, classrooms, or equipment, but the men who teach; and the good teacher is just about as valuable as the good engineer.

The writer does not hereby advocate the introduction of all the office details and routine. The student's time in college, and in the technical work, especially, is too short to admit of this, even if desirable. He has, for instance, never believed that it is the function of the engineering college to train men for the drafting-room of the manufacturer. The latter is usually willing to pay promising graduates to learn to make shop drawings. What the employer does insist on is that the graduate shall be promising, and will take hold promptly and in the right spirit. It is the duty of the college to instill the spirit of the office rather than introduce all its details. It must also look ahead and train for next year as well as for to-morrow; to start men in the direction of becoming good engineers, as well as good draftsmen.

What are the elements of the office which the instructor may introduce into the classroom? They are: the individual problem and responsibility for the same, self-reliance, accuracy, and efficiency. In regard to the first: when several men are put on the same problem, each is very prone to lean on the others, and it often happens that one strong man does all the thinking for a group of weaker men. The individual problem, which may be a definite part of a larger problem, is the condition actually met in the office. The student is then thrown wholly on his own responsibility. Of course, this places more work on the instructing staff, and is only possible for small sections, say ten or twelve at the most.

In regard to the second—self-reliance—the student should be made to get by himself, as much as possible, the technical knowledge needed to tide him over the difficulties and solution of his problem; and to go to the instructor only in matters requiring judgment and experience. The latter, of course, will watch and check him constantly, and not

Mr. Constant. let him go too far astray. He will always seek to develop the student's self-reliance and ability to meet his own difficulties. Some of these difficulties, and the larger aspects of the problem will be discussed by instructor and students in the frequent get-together periods, commonly called seminars.

Finally, the student will be held to a certain degree of proficient achievement and accuracy. His work will be turned over to another student, perhaps in another section, to be checked completely, and finally to the instructor, who will have his own complete notes. This is probably very similar to Dean Raymond's plan. The writer will have an opportunity to put it into operation in his own department during this coming year.

It is hoped that this plan will stimulate the interest and develop the best powers of the students. The instructor must stand ready to judge his men as impersonally and critically as does the chief draftsman or engineer employer, and to insist on a high grade of work, after making due allowance for beginners. It is impossible to grade a student, with this plan, by marks or examinations, but solely by the quality of his work from day to day, and the way in which he takes hold of it. When the student shows a grasp of the subject and promise of future proficiency, but is slow and falls behind in quantity of work executed during the term, he should, without being marked down, simply be required to spend a longer time on his task. One of the evils inherited from the academic college is the class system, which assumes that all men are equally quick in perception and performance. The writer is inclined to agree with the author that the engineering college might well utilize all the months of the year, or, at any rate, that every subject should start afresh with each term, thus giving the student the opportunity to drop out and re-enter at any term point, as he may wish. The student who does not complete his work in one semester may continue with it into the next, getting his credit when he has finally satisfied his instructor that he has acquired the requisite proficiency in the subject. In like manner, he gets his certificate when he has finally completed the course, whether it is in three or in five years.

Another strong reason for this greater flexibility in student movement is that it readily permits him to acquire practical experience in shop, field, or engineer's office during his course. In the German technical schools, in all but the *"bau construktionen"* courses, twelve months in some industrial shop or laboratory is required before the student is given his certificate of *"diplom-ingenieur."* If the object of the engineering college is to train men for engineering work, and this is to be done by placing them as early as possible in the engineering atmosphere, it is a distinct aid to the teacher to have his students go forth and find out by actual contact what this atmosphere is like.

Every instructor knows that the men who have thus interjected a year or two of practical experience into the middle of their course return more earnest, full of purpose, and desirous of making the rest of their time in college count for the most, than if they had had no such arousing experience. Dean Schneider is accomplishing this in one way, at Cincinnati, and the Germans in another, by actually forcing their men to get this outside experience, and all colleges should make it fairly easy for their men to drop out occasionally, if they so will. Those who are tempted to remain out permanently either will have good reason for doing so, or, as the author suggests, are such as the college and the profession can afford to lose. Mr. Constant.

Summing up the foregoing points:

1st.—The engineering colleges are confronted with new conditions, not of their own making, but due to the economic and social trend of the times.

2d.—The conditions can be controlled somewhat from without by diverting some of the stream of young men now entering the engineering colleges into artisan and craftsmanship schools, and by elevating, in the public estimation, the dignity of such manual occupations.

3d.—After such a natural sifting, the colleges might well lower their entrance requirements, so as to make it reasonably certain that no earnest and promising young man is excluded.

4th.—The college should seek to train men for engineering work such as they will actually find it when they leave its doors. This can be done by a proper choice of curriculum, by the selection of wise teachers, experienced both in the practical side of their specialty and in the best means of presenting it to meet the needs of their students, the latter through the creation of the office atmosphere, at least in the upper technical work.

5th.—The processes of elimination of the unfit should be effective and rigid. Only such as an employer would care to retain and develop should be permitted to graduate.

Finally.—Practicing engineers and manufacturers should be willing to co-operate with the colleges, as far as possible. This point, however, need not be emphasized, for it is the general experience of the colleges that the former are in sympathy with their work, and co-operate cheerfully whenever a call is made upon them.

Arthur B. Green, Jun. Am. Soc. C. E. (by letter).—For the purpose which Mr. McCullough has in view—the education of future engineers—his curriculum seems to be well designed. If a man who is "on to his job and can make good" must be accepted as an educated man, certainly the limitations of Mr. McCullough's discussion of education are justified, and he has completely solved the problem which arises out of that assumption. He has shown how to "manufacture," as he terms it, a perfect assistant. His product would contain exactly the Mr. Green.

Mr. Green. right ingredients of knowledge, proportioned according to exactly the right formula, and, above all, every safeguard would be provided that it might contain nothing superfluous. It would be an accurately standardized article. Each employer would feel the utmost security that, from the very first day, this ideal assistant would return to him the full value of its price, and if, in any case, there were doubt about it, the employer could, possibly, turn it over to the United States Bureau of Standards for quantitative analysis, and have its ingredients properly checked. Standard specifications are actually called for, to be prepared by employing engineers and handed to the schools, and the schools may be put on contract to furnish only product fulfilling these specifications. Indeed, when employers begin paying the expenses of these "engineer factories," perhaps this ideal can be realized.

It would be too bad to take any of this very seriously. It is not enough that a professional man shall be "on to his job and make good." Nothing more than that is required of skilled laborers or trade mechanics; the engineer must be educated—school-educated, self-educated, or, preferably, both. Education, according to Webster, is defined thus:

"The totality of the qualities acquired through individual instruction and social training, which further the happiness, efficiency, and capacity for social service of the educated."

Qualities make up education, not knowledge. Moreover, these qualities must "further" certain things about the educated, so that education is not a possession that is complete and perfect according to any specifications, but it is to grow and advance. Inasmuch as so many discussions of engineering education have gone wrong with their conceptions of the subject, these two points ought to be taken up more fully, to see what are the consequences.

In the first place, then, knowledge is not education. Just what subjects are studied by the one being educated is a secondary matter; the chief concern is that the study shall be inspired and directed in such a way as to develop qualities which further happiness, efficiency, and capacity for social service. Many great engineers there have been, and there are, who never studied a technical subject in school, and one electrical engineer of great note received all his schooling in literature and the classics. He is educated, nevertheless. He has the qualities of education at such potential that they are to be measured almost by the kilowatt. It is the function of technical schools, no less than of universities, so far as they can, to help along the development of such men as this. Their prime duty is to educate. Should they fail in that, the excellence of their curricula could not redeem them.

In insisting, therefore, on this distinction between education, on the one hand, and instruction, on the other, there is no quibble. On the contrary, it is definitely harmful to confuse the two. In an employer

looking over a candidate for employment it means laying the emphasis on just the wrong thing, and considering what he has, instead of the power he has to obtain more. From the business standpoint, it means reckoning immediate profits as more important than ultimate strength, both for the individual employer and for the Profession at large; and it also means, regarding the assistant not as a future engineer but as a piece of equipment. Mr. McCullough has been more frank than most others who take this attitude, and just as negligent of the consequences. The moment that the assistant to the engineer becomes standardized as a piece of apparatus for specific uses, immediately he is a trade mechanic and not an engineer. All he has to sell are his contents. He is designed to fit specific needs, and finds place only where those needs exist. Always working at the same manner of problem, he is unfitted to undertake anything new or to advance; his price is soon fixed accordingly, and he is a commodity. He has then entered, not the Engineering Profession, but the Engineering Trade. There is such a trade. Its members are somewhat learned and considerably skilled, but hardly educated. It is unfair to the engineering recruit to require him to fit himself for this trade and no more, to put him in competition with those who devote their lives to the trade, and to hazard his future on the outcome. It is unfair to the Engineering Profession thus to force its more highly endowed and better trained recruits into this trade, and so, temporarily at least, out of the Profession. That is a very bad use of opportunities. Mr. Green.

From what has been said so far, it may be gathered that the intention is to underrate the value of special knowledge in engineering. Possibly so. Yet, is this not true: Wide knowledge as well as the qualities of education are required that a man may do more than be "on to his job and make good"? One of the most absurd mistakes is to presume that there is any harm in the student learning beyond his needs. Learning gives power to learn. The value of it is not so much the storage in the mind of certain information, but the development of power. It is not undesirable, but absolutely essential, that the well-trained student be guided, while he may be, carefully in problems with which the world would not trust him for years after. He will be ready for them, or others like them, at the earliest possible date, and therein is strength for the Profession.

Secondly, and lastly, education is a process of growing, not a fixed attainment. If engineers are to employ men with a start toward education, they will not get the finished product. The work of finishing has only begun, and must continue. Who, then, is to carry it on? Employer and employed must carry it on. Sometimes it seems as if employers were trying to dodge this elementary idea of growth in young engineers purposely, in order to escape the necessity of doing anything toward it themselves. They try to unload upon the schools the whole responsibility for the education of coming engineers, and to assume

Mr. Green. that, once they have told the schools what they want, they have washed their hands of the matter.

It takes only a moment's reflection to see that this is not only an unpractical stand, but otherwise a harmful and absurd one. The burden of the education of young engineers is not on the schools at all, but on the employers. They cannot escape it, if engineering is to be a Profession. It is the highest duty of every employer to study his assistants, to discover their capacities and adaptabilities, and to develop to the fullest every potentiality. This is the highest duty, because the employer owes it, not only to the assistant and to himself, but still more to the Profession at large, and most of all to the public. Both the Profession and the public benefit vitally by having the best engineers for the world's engineering. The employer is the one to furnish them.

There are many employers who recognize this principle, and still attempt to escape the duty it imposes. A decided example is the head of a large contracting business, whose work from office to shoveller is most thoroughly organized. He recognizes it as a fundamental principle of scientific management that the employers must teach the employed the very best way of doing each single operation on the work, not only once but continuously; that the employers must learn to correlate the capacities and adaptabilities of the workmen with the requirements of the work so as to choose scientifically the best men for each job. Thus he spends large sums and much energy in picking out promising bricklayers, teaching them the best way to lay bricks, seeing that they do lay bricks in that best way, keeping account of their performance, and tracing thoroughly and rewarding the advancement of each individual bricklayer. He does the same for helpers, for those who load and unload his materials, for his office force, and so on. He looks upon all this as the duty, the extremely profitable duty, of the employer. He is right; but he omits his engineers. It is one of his axioms that fresh technical graduates are of no use; therefore he will have nothing to do with them until they have had three years or so of experience. In other words, his competitors may train them for him. Although he is ahead of his time in insisting on the harm that stand would do him if taken in regard to bricklayers, he is oblivious to the harm it does him in regard to engineers; but the real damage is greater than that. It reaches to the recruits as well as to the contractor, to the Profession more than to the job, and most of all to the public.

It is idle for any engineering employers to seek, as does Mr. McCullough, to unload upon the schools the whole burden of engineering education, for that belongs chiefly upon the employers themselves. Rather than study to appreciate the shortcomings of technical graduates, it would be fairer and more profitable to try their possibilities. When that is done, employers and teachers will become mutually helpful, as they ought to be, and a wise solution of the problem of engineering education may be surprisingly near at hand.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

AIR RESISTANCES TO TRAINS IN TUBE TUNNELS.

Disscussion.*

By J. V. Davies, M. Am. Soc. C. E.†

J. V. Davies, M. Am. Soc. C. E. (by letter).—Mr. Gibbs speaks of the influence of eddies on the air currents due to local obstructions along the sides of the tubes. The anemometer tests, mentioned in the paper, indicate that the velocity of the air is greatest in the center of the cross-section of the tube and least at the sides, where these obstructions are located, and, as they are not frequent, their influence on the ultimate air resistance cannot be great, although they do cause eddies and momentary fluctuations in pressure. Mr. Davies

In regard to Mr. Pegram's reference to the factor, L, in Formula 22, it should be noted that, in the body of the paper, L is given as the length between air exits, in feet, while, in the appendix showing the derivation of the formula, it is given as the length from front of train to air exit, and there it is explained that the constant in the formula is obtained from the average results in tests of resistance in tunnels 2 000, 4 000, and 6 000 ft. long, respectively. The formula, therefore, is at the best an approximation, and represents the average air resistant in a tunnel L feet long, instead of the resistance at a point L feet from the head of a train to an outlet. These results, obtained with the formula, therefore, will be too high at one end and too low at the other. The air that immediately follows a train is at a very high velocity, but it decreases quite rapidly as the train departs, while the air ahead of the train has a very much higher velocity for a much longer period, and, therefore, the distance ahead of the train was used in the derivation of the formula as having a more controlling influence than that in the rear.

* Continued from August, 1912, *Proceedings*.
† Author's closure.

Mr. Davies. With respect to Mr. Pegram's suggestion that the relative areas of the tube and the train should be introduced in the formula: The relation of the area of the car to the area of the tunnel is taken care of in the empirical constant in the static resistance formula, which may vary with a change in the ratio between the area of car and area of tube.

Mr. Churchill states that the air resistance decreases rapidly after the air is put in motion. This statement, when applied to a tube tunnel, is correct to a very limited extent only, the very fundamental principle of fluid friction indicating the contrary, as the resistance to air passing through a duct will increase as the square of the velocity. The tests described in this paper indicate that it does make a difference whether a tunnel is 1 000 or 5 000 ft. long, particularly where high velocity is used. Of course, it is not claimed that these tests justify the acceptance of the results given in Fig. 4 when applied to a tunnel 8 miles long, this being given only to indicate the need of further experiments before a project for a tunnel of this length, for high speeds, should be undertaken.

In regard to Mr. Churchill's reference to the observations in the City and South London tunnel, which indicate that very little ventilation was secured by the train: It is likely that this is caused by the slow movement of the trains and the lack of proper air outlets. As the experiments in the Hudson and Manhattan tunnels indicate, the fans could be dispensed with entirely if it were possible to have sufficient inlets and outlets to the surface, and, in fact, portions of these tunnels are entirely ventilated by the piston action of the trains, the fans being installed only for emergency use in case of fire, or "blow-out," causing a large volume of smoke with no trains in motion to expel it. The value of the piston ventilation is verified by Mr. Gibbs' tests in the Pennsylvania tunnels.

It appears to the writer that there are two matters which are of interest enough to refer to in connection with this subject, although, strictly speaking, they are only akin to the subject; they are the personal sensations felt by employees within the tunnels and by passengers riding in cars within the tunnels, due to the flow and resistance of air caused by the movement of trains. Immediately after the commencement of operation of the Hudson and Manhattan Railroad it was found by employees that there was little need to watch for the headlights of trains, as they could tell instinctively by the movements of the currents of air when trains were approaching, and by the velocity of the currents the nearness of the train. Taking one of the downtown tunnels as an illustration: it is immediately apparent to a person standing in the tunnel, near the Pennsylvania Station, Jersey City, the moment a train enters the tube under Fulton Street, New

York, or, in the reverse direction, to an observer at the end of the tube at Cortlandt Street, New York, the starting of a train from the Pennsylvania Station, Jersey City. These points are more than one mile apart. To an observer, the movement of air is very gentle at first, but as the train comes nearer the velocity obviously becomes greater, up to the point of immediate approach of the train. This is so marked that at junction points employees instinctively know, from the movement of the air currents, from which direction a train is approaching. Mr. Davies.

The influence on passengers riding in coaches is a matter which caused very considerable comment by commuters on the Long Island Railroad immediately after the opening of the Pennsylvania tunnels. The initial schedule of operation called for very high speeds through these tunnels, and trains entering from the Long Island City approach, at high speed, on a descending grade into the tunnels, would strike the portal at high velocity with an initial pressure of air which was found to be very troublesome to the ear-drums of those riding in the trains. The writer has been informed that, in consequence of this, the initial operating speeds have been quite materially reduced when trains enter the portal. At the speeds usually operated in the Pennsylvania tubes, and also in the Hudson and Manhattan tubes and the Interborough's Battery tunnel to Brooklyn, it has been found from repeated observations that there is a pressure of $\frac{1}{10}$ in. of mercury, by barometric reading, under ordinary conditions of operation, and that this pressure produces a most distinct sensation on the ear-drums of passengers. There is a peculiar fluctuation of these air pressures at points of passing outlets, such as approaching a vent shaft or enlargement. The pressure will bank up with a moving train to $\frac{1}{10}$ in. above the normal, and, immediately before passing such an outlet, will drop to slightly below normal, indicating a partial vacuum at the rear of the train, and then again a sudden ascent of pressure after the outlet, or enlargement, is passed, which is equivalent to again entering the portal of the tunnel. This fluctuation causes sudden "blows" (so to speak) upon the ear-drums, which are probably not noticed by those in the habit of traveling daily in the trains; but, to strangers or those who are not accustomed to such daily traveling, it is distinctly unpleasant. This physiological effect indicates very markedly to the observer the principles of air resistance which are outlined in the paper.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

NOTES ON BRIDGEWORK.

Discussion.*

By W. J. C. Howalt, Assoc. M. Am. Soc. C. E.

W. J. C. Howalt, Assoc. M. Am. Soc. C. E. (by letter).—The author says: Mr. Howalt.

"It is stated in the textbooks on applied mechanics that a continuous beam on three supports can be dealt with as two separate beams fixed at the intermediate point and simply supported at the ends."

The foregoing statement holds good only when the following conditions are fulfilled:

1. The two spaces must be equal.
2. All three supports must be on a straight line.
3. Cross-sections of the beam symmetrical around the center support must be equal.
4. The loads must be arranged symmetrically around the center support.

Conditions 1 and 3 are fulfilled in the author's case. Condition 2 is assumed to exist, but, when settlement is feared, it is quite advisable to consider what effect this may have. Condition 4 is not fulfilled at all, and renders the whole deduction void.

Apply a load, as in Fig. 1, and call the supports *A*, *B*, and *C*, as indicated in Fig. 2. The reaction at *B*, as found by the author, happens to be correct. The reaction at *A* will be $\frac{W\,b}{a+b} - \frac{1}{2}\,Z$, and at *C* $= -\frac{1}{2}\,Z$; this, being an uplift, must be counteracted by anchorage.

* This discussion (of the paper by S. Vilar y Boy, Assoc. M. Am. Soc. C. E., published in *Proceedings* for August, 1912, but not presented at any meeting), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Howalt.

On page 800* the author accounts for the total external load in the reactions at A and B (his equation, $X + Y = W$). It must be assumed, therefore, that no anchorage is applied at the C end of the beam, but this, in turn, will change the continuous beam to a simple one supported at A and B, with an overhanging end on which no load acts, and for which the formulas are not applicable.

Now look at Fig. 3, which is supposed to be a diagram showing the loading for maximum bending moment for both simple and continuous beams; in neither case is this diagram correct.

In the first case assume the train coming in at B and stopped when the 12-ton axle is 4.69 ft. from B. We will then get a maximum moment of 42.07 ft.-tons against the author's "maximum" moment of 38.7 ft.-tons.

Regarding the second case: The writer will only refer to Fig. 2. This position of the load gives a moment at $B = 49.92$ ft.-tons against the author's "maximum" moment of 30.00 ft.-tons. In both cases the formula for moment, M, on page 802,* was used, which formula, by the way, is wrong.

The author makes no distinction between reaction and shear. With loadings as in Fig. 3 and with continuous beams, the maximum reaction is 19.7 tons, but the maximum shear is only 18.55 tons. He should explain why he uses the same position of the load for finding both maximum moment and maximum shear.

* *Proceedings*, Am. Soc. C. E., August, 1912.

MEMOIRS OF DECEASED MEMBERS.

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

ARTHUR POWIS HERBERT, M. Am. Soc. C. E.*

DIED JUNE 16TH, 1912.

Arthur Powis Herbert was born in Philadelphia, Pa., on August 30th, 1855. He began work as a Rodman on the Colorado Central Railroad in 1872, and was also employed on the Union Pacific and Utah and Northern Railroads. In 1876-77 he was with the American Dredging Company, of Philadelphia, and in December of the latter year went to Brazil with the ill-fated Collins Expedition on the construction of the Madeira and Mamoré Railroad. On his return, in 1879, he was employed on the engineering force of the Pennsylvania Railroad as Assistant Engineer on the Valley Creek Improvement, and in charge of the Coatesville and Pomeroy Improvement.

He was one of the railroad pioneers of Mexico, going there in November, 1880, under the late W. C. Wetherill, M. Am. Soc. C. E., Chief Engineer, as one of the engineers of the Mexican National Construction Company (Palmer-Sullivan Concession) on location and construction across the mountains from Mexico City to Toluca. In 1886, he was for about a year in charge of the extension of the Oroya Railroad at Cerro de Pasco, Peru. Returning to Mexico, he had charge of the location of the Guadalajara Branch of the Mexican Central Railroad, under the late John E. Earley, M. Am. Soc. C. E., and also of parts of the San Luis Potosi and Tampico Divisions.

Later, he again entered the employ of the Mexican National Construction Company, for which he made various surveys and locations and finally, under the late H. H. Filley, M. Am. Soc. C. E., Chief Engineer, located and built the portion of the Colima Division of the Mexican National Company's line between Armeria and Colima. Subsequently, he was made Superintendent of this branch, holding the position for about twenty years or until it was taken over by the Mexican Central Railroad Company and merged into one of the other divisions of that Company.

Mr. Herbert was well known in Mexico and South America. He was a careful and successful locating engineer, showing such rare judgment in his choice of lines that his final location seldom varied more than a few meters from the preliminary. His sterling worth was universally appreciated by his superiors, by whom he was trusted implicitly; his genial disposition and his strict and impartial justice

* Memoir prepared by Caspar Wistar Haines, M. Am. Soc. C. E.

won for him the friendship, admiration, and high esteem of associates and subordinates.

He died suddenly, from apoplexy, on June 16th, 1912, at his home in Colima, Mexico, leaving a widow and one daughter.

Arthur Powis Herbert was elected a Member of the American Society of Civil Engineers on September 5th, 1888.

JAMES BREADING HOGG, M. Am. Soc. C. E.*

Died June 4th, 1912.

James Breading Hogg, first son of John T. and Caroline (Austin) Hogg, was born on December 15th, 1857, near Prittstown, Bullskin Township, Fayette County, Pa. Soon after his birth his family moved to what was then New Haven, on the west bank of the Youghiogheny River, now known as the West Side, Connellsville, Pa., where he spent the first twenty-seven years of his life.

He was graduated from Lafayette College, Easton, Pa., with the degree of Civil Engineer in 1881. He was a member of the Lafayette College baseball team, and also played on one of the early baseball teams of his home section.

After graduation he commenced his engineering career, under the late Joseph H. Paddock, M. Am. Soc. C. E., as an Instrumentman on the construction of the Pittsburgh, McKeesport and Youghiogheny Railroad, between Layton and New Haven, Pa., and the Dickerson Run Branch of the same railroad. He was also on the construction of a highway bridge across the Youghiogheny River at Dawson and a railroad bridge over the same river at Broad Ford. At that time the writer was also a member of the Engineering Department of that railroad, and it was then that his acquaintance with Mr. Hogg began.

In 1886 Mr. Hogg went to Puget Sound, where he served as Assistant Engineer on the Cascade Division of the Northern Pacific Railroad, having charge of 10 miles of heavy construction work. On its completion he was associated with the City Engineer of Seattle, and for two years had charge of that office. He was Assistant City Engineer of Seattle at a time when the city was being laid out along progressive lines which necessitated extensive changes in grades and street lines. The beauty of Seattle's streets to-day is due in a large measure to the intelligent and able engineering work of Mr. Hogg.

Later, he was Chief Engineer of the Port Townsend Southern Railway, and for two years directed the exploration, location, and construction of its line. Then he became Assistant Engineer for the Oregon Railroad and Navigation Company, and constructed 15 miles

* Memoir prepared by Emile Low, M. Am. Soc. C. E.

of its track. He was also employed in the Washington State Engineering Department at various times.

During a period of industrial depression, when railroad construction was suspended, Mr. Hogg was for two years County Treasurer of Jefferson County, Washington. He returned to the East in 1900, and entered the service of the H. C. Frick Coke Company, at the time of the spectacular entry of Andrew Carnegie, the ironmaster, into the railroad world. Here he planned a railroad through the Connellsville coke region which connected all the plants of the H. C. Frick Coke Company. He also made explorations across the Allegheny Mountains for Carnegie's proposed railroad to the Atlantic Coast. Later, he was Engineer in Charge of the surveys for the same company and became Division Engineer of the Northern coke field.

In 1902 Mr. Hogg was transferred to the Frick interests in West Virginia, and made topographical surveys in the Tug River District of McDowell County in that State. Completing that work, he returned to Fayette County and opened an office in Connellsville, Pa., for general consulting practice. He also maintained offices in Uniontown, Brownsville, Scottdale, and Pittsburgh, all in Pennsylvania. When death halted his work he was serving his fourth term as County Engineer of Fayette County. He was also Borough Engineer of Connellsville, Scottdale, and Everson, and previously had been Borough Engineer of Coraopolis, Bentleyville, and West View. In addition, he made comprehensive sewer plans for Connellsville, Uniontown, Scottdale, Brownsville, West Brownsville, and South Brownsville.

Mr. Hogg was one of the best known civil engineers in Western Pennsylvania, and ranked high in his chosen profession. He lived an active life, gained a wide experience in all departments of engineering, and built up probably the most extensive engineering business in that section of Pennsylvania. Realizing that municipal sanitation was to become an important factor in that State he specialized diligently in that line of work. Although stricken in the prime of manhood, he had already attained a degree of prominence as a sanitary engineer that caused his counsel to be sought by many municipalities in Western Pennsylvania.

His first work was the preparation of a comprehensive sewer plan for Connellsville, and the map which he completed was one of the best ever submitted to the engineers of the Pennsylvania State Board of Health. He was frequently called to Harrisburg to consult with the State sanitary engineers, and his judgment was highly respected.

Mr. Hogg had great faith in the future of Connellsville. He believed, not only that the town would grow, and grow rapidly, but was firm in his conviction that eventually there would be a civic awakening resulting in Connellsville being not only a prosperous city, but a more desirable place in which to live. He believed in the "city beautiful"

movement, not in the sense that has made such a sentiment a reproach, but that a serious, concerted effort should be made to beautify the streets, the houses, and surroundings. He believed that Connellsville was neglecting a wonderful opportunity in failing to capitalize its river, to beautify the river front, and, if possible, dam the stream and create of it something useful to the community. He could not understand why such a manifest advantage was neglected, or why more interest was not exhibited in the movement to utilize the river (Youghiogheny), which he, with a few others, fully realized.

Mr. Hogg did not hesitate to back his faith with works. The beautiful East Park Addition to Connellsville will ever be a monument to his belief in Connellsville and his efforts to make it more attractive. For years "Hogg's pasture" had furnished grass for the cows of the community. Its hills rose beyond a steep ravine which seemed a perpetual obstacle to development. In 1904 Mr. Hogg determined to lay out a new addition. It was not primarily a land selling scheme. Mr. Hogg had other plans, and, in co-operation with the John T. Hogg estate, proceeded to make an investment that seemed out of proportion to the possible returns within a generation. An immense steel bridge was constructed from Baldwin Avenue across the hollow through which Connell Run winds its way to the river. Beyond that point Mr. Hogg set his engineers at work. A boulevard was laid out through the Hogg estate as far as the Reidmore Road. Once the lines were laid, the road was built and was one of the most attractive driveways in that section.

At the time the East Park Addition was conceived, Mr. Hogg's home was in Uniontown, although his engineering offices were in Connellsville. He immediately awarded a contract for the erection of a fine dwelling in the new addition. For a year or more it loomed alone in the heart of the barren pasture. Then came other residences, and before long East Park Addition had "arrived."

The lots were sold under certain building restrictions which encouraged the erection of attractive homes. Where six years ago there was but one home, and that occupied by Mr. Hogg himself, for he moved there soon after its completion, there is to-day quite a colony of attractive residences, and others are contemplated. With characteristic public spirit Mr. Hogg dedicated the boulevard to the public. This was named Will's Road, in memory of his brother, the late William A. Hogg.

Mr. Hogg was elected a Member of the American Society of Civil Engineers on October 3d, 1906. He was also a Member of the Engineers' Society of Western Pennsylvania, the Engineers' Society of Pennsylvania, and the American Mining Congress of Denver. Aside from his business, Mr. Hogg had time to give to other matters. He was a member of the Chamber of Commerce of Connellsville, and one of its Directors. He was also a member of the Uniontown Country

Club and the Laurel Club. In politics he was a staunch Republican, but in no way a politician. In religion he was an Episcopalian.

On December 22d, 1903, Mr. Hogg was married to Miss May Reid, daughter of the late William T. Reid. He is survived by his widow and also his mother and four sisters.

Mr. Hogg died on June 4th, 1912, and was buried in Allegheny Cemetery, Pittsburgh, Pa., where his father is laid at rest.

The following resolutions, passed by the Connellsville Chamber of Commerce, show the respect in which he was held:

"James Breading Hogg, member of the Chamber of Commerce of Connellsville, and one of its most valued, capable, and useful directors, was possessed of those sterling qualities which differentiate men of worth and achievement from those of sham and pretense; of those enviable characteristics which are the outstanding traits of men of mark and usefulness in a community's life; of that temper of mind and fulness of heart which drew others to him by the ties of friendship, respect, and esteem, and of that tenderness and gentleness which enshrine his memory in the most affectionate remembrance of those who loved and were loved by him.

"He attained conspicuous rank in his chosen profession because of thorough preparation, close application, fidelity to the interests he served, and the high ideals by which he was constantly stimulated and impelled.

"As one of Connellsville's First Citizens, the disinterested, zealous, public-spirited services he rendered in the development and advancement of the material interests, and the enhancement of civic virtue, of the whole community, entitle him to enduring honor and a people's gratitude.

"Proving his faith in the future growth and greatness of his native town, by his own works and efforts to arouse a civic awakening that would be active, progressive, and permanent, he set an example worthy of emulation by every citizen who should seek, as unselfishly as did he, to make Connellsville a better home for the present and all succeeding generations.

"As an earnest and sincere, though by no means adequate, measure of the Chamber's appreciation of the life services, character, and attainments; as a meed of praise of the deeds he wrought, and in highest respect, admiration, and regard for our lamented associate and co-worker, these sentiments are recorded, by order of the Chamber, this sixth day of June, A. D., 1912."

EDWARD HENRY KEATING, M. Am. Soc. C. E.*

DIED JUNE 17TH, 1912.

Edward Henry Keating was born at Halifax, Nova Scotia, on August 7th, 1844. He was the fourth son of William Henry Keating, Barrister, who, for many years, was Deputy Provincial Secretary of Nova Scotia.

* Memoir prepared by W. H. Breithaupt, M. Am. Soc. C. E.

Mr. Keating received his education at the Free Church Academy, Halifax, and at Dalhousie College. His studies had been directed with a view to taking up Architecture as a profession, but he was soon engaged in surveys, and studied Engineering under George Whitman, Civil Engineer, Provincial Government Engineer of Nova Scotia.

For three years Mr. Keating was employed on the surveys, location, and construction of the Truro and Pictou Railway, and was also engaged as Chief Draftsman of the Windsor and Annapolis Railway. At the close of 1867, he entered the employ of the Intercolonial Railway and was Assistant Engineer on some of the heaviest construction, remaining in this position until the road was nearly completed.

In the spring of 1872, Mr. Keating was appointed Division Engineer in charge of exploration on the Canadian Pacific Railway surveys, but resigned at the end of the same year to become City Engineer and Engineer of the Water-Works of Halifax. He was also Resident Chief Engineer of the Halifax Graving Dock, in its time the largest graving dock on the American Continent. He remained in Halifax until the end of 1890. During his residence there, he also designed water-works for Truro, Windsor, and Dartmouth, Nova Scotia, and for Moncton, New Brunswick.

For two years, 1891 and 1892, Mr. Keating was City Engineer of Duluth, Minn., where he designed extensive improvements for the water-works. He was then offered and accepted the office of City Engineer and Engineer of the Water-Works of Toronto, Ont., and soon reported on a comprehensive plan for the improvement of its water-works. This plan was considered too expensive at the time, but was afterward carried out. He did important improvement work in Toronto Harbor, and also in Keating Channel which was named after him. In 1898, Mr. Keating resigned as City Engineer of Toronto to become General Manager of the Toronto Street Railway, which position he held for six years.

In 1903, he was appointed Chairman of the Royal Commission to report on the construction of a graving dock for the City of Montreal. He also acted, at various times, as Expert Advisor on the water-works and sewerage systems of Ottawa, Hamilton, Victoria, B. C., etc.

In 1904, Mr. Keating went to Mexico where, for the greater part of two years, he was Engineer and Manager of the construction of the street railway and power and lighting plant of the Monterey Railway, Light and Power Company, and of the Monterey water-works and sewerage systems. Of these properties he remained Advisory Engineer for some years after his return to Toronto.

For the last six years of his life, Mr. Keating was active as a Consulting Engineer in general practice, being engaged for a greater part of his time as Arbitrator in valuation proceedings, for which

his sound judgment and absolute sense of justice admirably fitted him.

Mr. Keating was elected a Member of the Institution of Civil Engineers in 1878, and had been a Member of the Council since 1911. He was President of the Canadian Society of Civil Engineers in 1901, having long been a Member and Member of the Council of that Society. He was also a Fellow of the Imperial Institute, a Member of the Engineers Club of Toronto, of which he was one of the organizers, and of the Toronto and York Clubs.

In February, 1907, he was appointed a member of the Board of Examiners for Professional Degrees in Engineering, by the University of Toronto, and was Chairman of this Board until early in 1912, when, on account of ill health, he was compelled to resign as Chairman, though he continued as a member until his death on June 17th, 1912.

Mr. Keating was elected a Member of the American Society of Civil Engineers on June 7th, 1882.

WALTER SCOTT HANNA, Assoc. M. Am. Soc. C. E.*

Died July 4th, 1912.

Walter Scott Hanna was born in Lykens, Pa., on September 16th, 1879. He was educated at Pennsylvania State College and Lehigh University, from which latter institution he was graduated in 1902 with the degree of C. E.

During the summer vacations of 1898, 1899, and 1900, Mr. Hanna was employed as Chainman, Leveler, and Transitman on mining engineering corps of subsidiary companies of the Susquehanna Coal Company, at Shamokin and Lykens, Pa., which work proved of immense value to him in his subsequent work at the Massachusetts Institute of Technology, in lecturing on practical mine surveying and rock excavation.

From June to September, 1901, he was engaged as Chief of Party and Assistant to the Maintenance-of-Way Engineer of the Norfolk Division of the Southern Railway, devoting all his spare time during this period to the study of the operation of the Division. In June and July, 1902, Mr. Hanna was Assistant Instructor in Topographical Surveying at the Lehigh University Summer School, and had charge of several field parties. Later, from July to September, he was employed as Assistant to the Superintendent of Blast Furnaces of the Pennsylvania Steel Company, at Steelton, Pa., studying blast-furnace operations and steel manufacture.

* Memoir prepared by the Secretary from information furnished by A. F. Hanna, Esq., and from papers on file at the House of the Society.

In September, 1902, he was appointed Assistant Instructor in Civil Engineering at the Massachusetts Institute of Technology. While in this position, besides being engaged in outside work, he did post-graduate work in Civil Engineering at the Institute.

In June, 1903, Mr. Hanna was employed as Assistant Engineer with the American Pipe Manufacturing Company, of Philadelphia, Pa. While with this Company, he was appointed Resident Engineer of the reservoir, etc., then being built at Scarsdale, N. Y., and also assisted the Chief Engineer with the mathematical calculations for a new water meter.

During August and September, 1904, Mr. Hanna located a railroad at Lykens, Pa., for the Lykens Valley Coal Company. He then removed to Columbus, Ohio, where he was employed as Draftsman for the Board of Public Service, on designs for a sewage pumping station, clear-water reservoir for a filter plant, etc. He remained in this position until January, 1905, when he was engaged as Engineer by Hugh MacRae and Company, Bankers, of Wilmington, N. C., on a land development project.

In November, 1905, Mr. Hanna returned to the Massachusetts Institute of Technology as a student and as Assistant Instructor in Civil Engineering, remaining until March, 1906, when he was appointed Assistant Engineer with the State Water Supply Commission of Pennsylvania, being engaged in investigating and making reports on water supply companies, projects, etc. He was also employed on work for the Pennsylvania State Board of Health.

In October, 1910, he was appointed Consulting Engineer with the J. B. Hogg Engineering Company, of Pittsburgh, Pa., but in January, 1911, he was compelled to retire, on account of ill-health. From that time until his death, on July 4th, 1912, Mr. Hanna made his home at Asheville, N. C. He is survived by his widow and one daughter.

Mr. Hanna was elected a Junior of the American Society of Civil Engineers on October 6th, 1903, and an Associate Member on June 5th, 1907.

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AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

TUFA CEMENT, AS MANUFACTURED AND USED ON THE LOS ANGELES AQUEDUCT.

By J. B. Lippincott, M. Am. Soc. C. E.

To be Presented December 4th, 1912.

Los Angeles is situated in a region where the annual rainfall is only 15 in., and where a water supply is requisite, not only for domestic necessities, but also for the beautification of grounds and all forms of intensive agriculture. The Federal census shows that the population of the city increased from 102 479 in 1900 to 319 198 in 1910, or 211%, which is the greatest growth in any of the larger cities of the United States during this period.

General Description of the Line.—The city relies on the Los Angeles River for its local water supply. This stream rises from the gravel beds of the San Fernando Valley, and is uniform in its flow. It has been completely diverted, and encroachments have begun on the underground waters of the neighborhood. What is known as a miner's inch in Southern California is equivalent to 13 000 gal. per day, or $\frac{1}{50}$ cu. ft. per sec. The right to a miner's inch of water of continuous flow in this locality varies in value from $1 000 to $2 000, and is a measure of the scarcity of local water supplies.

Instead of exercising its right of eminent domain, and attempting to condemn other streams in Southern California, which would result in the depletion of commercially tributary areas, the Board of Water

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

Commissioners adopted the broad policy of going to some distant source for its new supply, where the minimum damage would be done to others, and where the maximum quantity of water would be obtained. As the value of water is increasing very rapidly throughout California, it was decided to construct as large an aqueduct as the city could afford. It was found that the most available supply could be obtained from the Owens River, which, for a distance of 120 miles, drains the eastern face of the Sierra Nevada. There are 40 peaks in this crest, the elevation of which rises above 13 000 ft., Mt. Whitney being the culmination of the range, with an elevation of 14 500 ft. This is the source of the water supply. To the east are the Inyo Mountains, a lower range, blanketed by the high Sierra intervening between the Inyos and the Pacific Ocean, and barren of water crop.

Owens River discharges into Owens Lake, having an area of 64 000 acres, from which the annual evaporation loss is about 6.8 ft. The lake has no outlet and consequently is saline. The aqueduct is being built to deliver a continuous flow of 20 000 miners' inches, or 400 cu. ft. per sec., which is two-thirds of the evaporation loss from the lake. The diversion is made 35 miles above the lake, and the city has purchased 105 000 acres of land in Owens Valley, including both banks of the river from the diversion point to the lake.

The precipitation on the crest of the Sierra Range varies from 40 to 60 in., and is mostly in the form of snow occurring during the winter, the drifts accumulating in the high mountain gorges and melting with the approach of summer. These snowbanks, however, are of such extent that they last over from year to year and some of them have compacted into banks of ice. The high-water period occurs in June and July, low water extending from September through the fall and winter. The annual rainfall in the valley floor is only 5 in.

For the first 60 miles, the capacity of the aqueduct as designed is equal to the mean June flood of the river at the intake, or 900 cu. ft. per sec. This portion of the conduit discharges into the Haiwee Reservoir, which has a capacity of 63 000 acre-ft., with a maximum center height of dam of 91 ft. This Haiwee Reservoir will act as a regulator, from which a continuous flow of 420 cu. ft. per sec. will be drawn. In addition to the surface streams, a large Artesian basin

has been discovered in Owens Valley, from which ground-waters can be extensively drawn during years of drouth. In case this supply is insufficient, in the driest years and when the consumption of water approaches the full capacity of the aqueduct, the city can build the Long Valley Reservoir, which site it controls. This will have a capacity of 341 000 acre-ft., with a dam having a maximum height of 160 ft.

For the first 20 miles in Owens Valley the conduit is a dredged earthen canal. This part of the line is in the moist bottom lands of Owens Valley, which are saturated with water, and where the aqueduct will gain water. All other portions of the aqueduct are being lined with concrete. The 40 miles of the aqueduct immediately north of the Haiwee Reservoir, because of its large size, is an uncovered, but lined, canal. With the exception of the first 60 miles previously described, all portions of the aqueduct are lined and covered.

From the diversion point in Owens Valley, the line skirts the eastern base of the Sierra Nevada as far south as the Town of Mojave. It then crosses the western edge of the Mojave Desert, passes under the Coast Range, in a tunnel 26 860 ft. long, and then has three drops aggregating 1 842 ft., where power plants are being installed. Immediately above and below the first two power sites, with drops of 1 516 ft., reservoirs are being constructed.

The capacity of the aqueduct between these two power sites has been increased to 1 000 cu. ft. per sec., in order to provide for the variable power load. With these two reservoirs, it will be possible to increase the flow through the power plants during certain hours of the day to 1 000 cu. ft. per sec., and to regulate it back, below the power plants, to a continuous flow of 400 cu. ft. per sec. In other words, the power factor is taken at 40 per cent. Other large storage reservoirs are being built at the extreme southern end of the line.

The total quantity of cement required for the construction of the aqueduct is estimated at 1 500 000 bbl. Table 1 is a summary of the different classifications of work.

The conduit will deliver water at a point 25 miles north of the city, where the distribution system starts. On March 1st, 1912, 83% of the work was finished, and it is being completed within the estimated time and well within the estimated cost.

TABLE 1.—LENGTHS AND SECTIONS OF VARIOUS PORTIONS OF THE LOS ANGELES AQUEDUCT.

Classification.	Length, in miles.	Capacity, in second-feet.
Unlined canal	21	900
Open, lined canal	40	900
Haiwee by-pass	2	420
Covered conduit	98	420
Lined tunnels	43	420
Concrete flumes	0.2	420
Concrete pipe, 10 ft. diameter	2.8	420
Steel pipe	9.4	420
Power tunnels	8.8	1 000
Reservoirs	8.5	.....
Total length	233.7	.....

City Cement Plant.—The city has built a standard Portland cement plant on the Southern Pacific Railroad, near the center of the aqueduct line, at a place named Monolith, which is the brand name given to the cement. The mill has a capacity of 1 200 bbl. per day. The operation of the mill is successful, and the cost of producing the cement is reasonably low.

There are six other Portland cement works in California, the products of which are reliable and satisfactory. Apparently, however, there is a definite agreement among these manufacturers as to the selling price. It was not contended that the city could manufacture cement either cheaper or better than some of these larger plants; but the location of the city's cement plant on the line of the aqueduct eliminates 25 or 30 cents per bbl. in freight charges, and it was believed that the city could manufacture its cement on the line of the work at a price which probably would be lower than that for which manufacturers would sell their delivered product. Moreover, by having its own mill, the city is assured of deliveries at the rate required.

The Monolith cement mill is 14 miles from Mojave. A railroad has been built under contract with the city by the Southern Pacific Company, 140 miles long, northward from Mojave, along the line of the aqueduct; because of this special contract, however, freight rates on this new line are high, and amount to nearly 1 cent per barrel-mile.

Three tufa cement-grinding plants have been established on the line of the aqueduct, namely, Haiwee, Fairmont, and Monolith, extensive deposits existing at each of these points. Haiwee, where the northern tufa-grinding plant has been built, is 120 miles by rail

from Monolith, and 106 miles north from Mojave. The southern tufa-grinding plant, at Fairmont, is about 20 miles from the railroad and southwest from Mojave. Transportation charges from Monolith to both Haiwee and Fairmont amount to 90 cents per bbl. As the tufa cement process converts 1 bbl. of standard cement into 2 bbl. of tufa cement, there is saved in transportation charges alone about 45 cents per bbl. on the tufa cement product.

Tuff or tufa is a volcanic, pumiceous rock composed of minute particles bearing indications of having been laid down in water and partly consolidated. Sometimes tufa is a calcareous deposit, but that used for the manufacture of the cement described herein is of volcanic origin. It is of a grayish or creamy color and has a low specific gravity when in rock form; when pulverized, the powder has a specific gravity of 2.2.

According to Dana's "Manual of Geology," puzzuolana is a light-colored tufa found near Rome and elsewhere in Italy, and is used for making hydraulic cement. Puzzuolana is a local name, and tufa or tuff is the geological term. Samples obtained from Italy through the Consular Service in Rome show the puzzuolana to be a light purple-colored fragmental material having somewhat the appearance of volcanic ashes, and unconsolidated. Its analysis is given in Table 2. The tufa used on the Los Angeles aqueduct resembles the German trass, used in the manufacture of the German trass cements.

TABLE 2.—ANALYSIS RECORD OF VARIOUS TUFAS.

No.	Description.	Date made, 1912.	SiO_2.	R_2O_3.*	Fe_2O_3.	Al_2O_3.	CaO.	MgO.	SO_3.	Loss.	Total.
220	Monolith tufa cement..........	1/15	35.34	11.89	......	......	41.05	3.04	......	8.52	
229	Fairmont tufa, middle of quarry	1/29	71.36	16.15	......	......	2.86	trace	0.116	3.22	
230	Fairmont tufa, south side of quarry..........	1/29	70.06	17.11	......	......	1.81	"	0.035	6.51	
233	Monolith tufa......	2/5	68.26	17.10	......	......	2.60	......	......	......	Alkalies.
215	" "	....	69.46	13.89	2.52	11.37	1.80	2.95	0.429	6.28	4.7
...	Hawaiian lava.....	1/15	51.98	18.75	2.90	15.86	9.57	5.61	......	......	
262	Italian tufa........	3/25	42.36	28.35	......	......	9.15	0.54	0.56	13.68	
263	" puzzuolana	3/25	45.68	30.09	......	......	11.95	3.76	0.56	6.30	

* $R_2O_3 = Fe_2O_3 + Al_2O_3$.

All the tufa along the aqueduct occurs in beds 100 ft. or more in thickness, and shows distinct lines of stratification, indicating that

it was laid down in water. The beds near Haiwee are in the immediate neighborhood of ancient volcanoes and lava flows. Ancient craters exist in the desert between Fairmont and Monolith. The Fairmont tufa is the purest of the three deposits. It is fine-grained or comminuted, and portions of the bed are entirely free from pebbles or other foreign matter. The Haiwee tufa is quite free from pebbles, but it contains some mica, which makes the grinding slower. The tufa rock at Monolith is more compact, and has a specific gravity of 1.97. It contains a good many pebbles of a flinty character, and occasionally granitic, which makes the grinding slower, but does not affect the quality of the product. The three tufas used on the aqueduct, when blended with cement, are much the same in strength, the average monthly tests showing greater strength first at one mill and then at another.

Silica cement is a term applied to mixtures of cement and silica, usually in the form of sand, ground together in a dry state to a greater fineness than the Portland cement. This mixture is then used with sand and gravel, as in the ordinary method of preparing concrete. The proportion of pure cement is reduced without a corresponding reduction in the strength of the concrete. The voids in the sand are filled with the ground sand, the gradation of the concrete aggregates being thus carried one step farther. This silica is not soluble with lime, and the cement is improved by the regrinding. In 1899 the United States Geological Survey made investigations of sand cement along the upper portion of the Gila River, in Arizona. The tests were made by laboratory grinds and are shown in Table 3. The silica used was a rock known as pearlite, which is a form of rhyolite, from the Butte dam site, and also a quartzite from the San Carlos dam site.

All the tests were made with the same sample of cement. The mixtures were made by weight. It will be noted in Table 3 that, in the case of the silica cements Nos. 1 and 2, all the cement used was passed through a 200-mesh screen. This was unfortunate, because it gave an abnormal fineness, and consequently an undue strength is shown; but, in the case of Nos. 5 and 6, the straight cement was treated in a similar manner, so that comparisons are possible. The sands used were standard in size, but were not standard testing sands.

Comparisons of Test No. 3 with Test No. 5 show the effect of fine grinding on standard cements.

TABLE 3.—RESULTS OF TESTS OF PORTLAND SAND CEMENT.

No.	Material.	Sand.	Portland cement to sand.	PERCENTAGE OF FINENESS.			Per-centage of water.	STRENGTH, IN POUNDS.	
				50 mesh.	100 mesh.	200 mesh.		7 days.	28 days.
1	Colton and Butte Pearlite, 1 to 1.....	2	1 to 5	0.00	0.00	0.00	10	80	300
2	Colton and San Carlos Quartzite, 1 to 1.......	2	1 to 5	0.00	0.00	0.00	10	90	345
3	Colton regular.......	2	1 to 2	0.42	7.20	33.20	10	170	385
4	Colton regular.......	3	1 to 3	0.42	7.20	33.20	10	140	240
5	Colton fine ground..	2	1 to 2	0.00	0.00	*	12	370	465
6	Colton fine ground..	3	1 to 3	0.00	0.00	*	12	170	260
7	Colton and Butte Pearlite, 1 to 1.....	3	1 to 7	0.00	0.00	0.00	10	75	155
8	Colton and San Carlos Quartzite, 1 to 1................	3	1 to 7	0.00	0.00	0.00	10	55	185

* Some left on 200-mesh screen.

Since 1903 the writer, from time to time, has investigated the possibility of blending tufas with cement. He identified the Haiwee deposit of tufa, and shipped some of it to the laboratory at the Monolith cement mill. This led to the identification of the ledge of tufa in the immediate neighborhood of the mill and also to the discovery of the Fairmont deposit. The tufa was first ground with cement experimentally in the laboratory, and showed satisfactory tests. The experiment of mixing the ground tufa with slacked lime, without any cement, was also tried, and it was found that this would set under water and slowly become hard, but it checked in drying in the pats. This hydraulic property indicated the solubility of the tufa in hydrated lime and its power to combine with the excess lime in the cement. This does not occur with silica cement.

A mill run was then made at Monolith, and a length of several hundred feet of canal was lined with tufa cement concrete, in order to observe its working conditions in the field. As this proved satisfactory, it was decided to build tufa regrinding mills at both Fairmont and Haiwee.

The Fairmont mill consists of a Climax jaw crusher which breaks the tufa to about a $1\frac{1}{2}$-in. size. It is then carried to a No. 8 Krupp ball mill, where it is ground to pass through a 20-mesh screen or

finer. This ground tufa is then blended in equal parts by volume with the standard cement. This blend is then conveyed to a Gates tube mill, 6 by 16 ft. in size, and the tufa and cement are ground together to a fineness of 90% or more, passing through a 200-mesh sieve. The cost of the plant was $27 000. The grinding is much freer during the dry summer months in California than during the wet winter weather. A little moisture in the tufa seriously reduces the product, as it coats the pebbles, thus lowering their grinding efficiency. At the Fairmont mill it was found that, under natural conditions, from 1 200 to 1 500 sacks of tufa cement could be ground per 24 hours. By arranging crude drying devices and driving off the moisture in the tufa with a slow wood fire, this output was increased to from 1 800 to 2 000 sacks per day. In both the Haiwee and Fairmont quarries, the capacity of the ball mill is 40% in excess of the capacity of the tube mill.

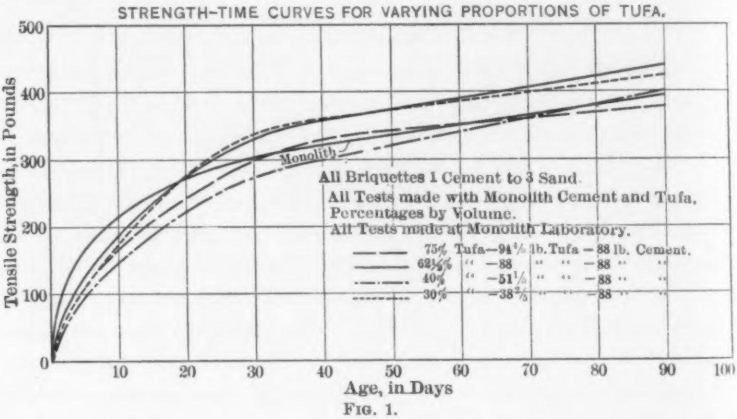

FIG. 1.

Table 2 shows the analysis of various tufas, including the Italian samples and Hawaiian lavas. Tests were made in the laboratory by mixing 4 lb. of tufa, 4 lb. of cement, and 1 lb. of slacked lime. This gave a result from 50 to 75 lb. per sq. in. stronger than when only tufa and cement were used. Subsequently, some mill runs were made with lime added in this manner, but they did not confirm the laboratory tests with lime, and the practice of adding the latter has not been continued. Mill runs were made with varying proportions of tufa at the Fairmont plant, and the results, with different percentages of tufa, are given in Fig. 1. These tests are not fully sustained by later mill and

laboratory tests, in that the higher percentages of tufa at first shown were relatively too strong. Possibly the mill run was not long enough to establish thoroughly the various ratios in the output. Finely ground Monolith tufa mixed with thoroughly hydrated lime in the ratio of 75% of lime to 25% of tufa was made into briquettes with sand in the ratio of 3 to 1. The briquettes were left in a damp closet for 28 days and then immersed in water, and gave the following strengths:

7 days.	28 days.	3 months.	6 months.
40 lb.	80 lb.	70 lb.	225 lb.

The briquettes were softened on the surface by the action of the water.

TABLE 4.—TYPICAL SAND BRIQUETTE TESTS, AT MONOLITH, CAL.

Briquette number.	Date made.	Percentage of water.	Tensile Strength.						Brand.	Percentage of tufa.	Fineness.		Setting Time.				Boiling test, 6 hours.
			3 days.	7 days.	28 days.	3 months.	6 months.	1 year.			100 M.	200 M.	Initial.		Final.		
													H.	M.	H.	M.	
20 Tests	1910 Nov.	9	114	203	408	460	518	577	Tufa Monolith Fairmont	50	98	89	2	19	5	49	O. K.
20 Tests	Dec.	9½	104	160	363	460	477	511	Tufa Monolith Haiwee	52	98	91	3	00	6	12	O. K.

Table 4 shows typical tests of the regular mill runs of the tufa cements, as manufactured at Fairmont and Haiwee, and as used in the construction of the aqueduct. A feature to note is the constant growth in strength of the samples. They are occasionally below the standard of strength required by the American Society for Testing Materials for 7 days (from 150 to 200 lb.), but are above the standard for 28 days (from 200 to 300 lb.). As far as tested, the tufa cements manufactured on the aqueduct uniformly show this growth in strength with age, and in this respect are superior to the tests for strength in straight cement, which often show a loss after 28 days. Tests made by the Santa Fé Railway indicate that this loss in strength of straight cement continues, as far as observed, through a 5-year period, in four out of five brands tested.*

As tufa cements are high in silica, and as the silicates of lime are the more enduring but slower portion in the cements, this growing

* *Engineering News*, March 14th, 1912.

strength in tufa cement is quite rational. Straight cements which are slow in hardening show the greatest ultimate strengths, and a high 7-day test is regarded with suspicion.

Briquettes made of pure tufa cement without sand do not show as great strength as neat cement, but as cement is not used in practice in this form, it is relatively unimportant. The leaner the mixture in sand briquettes, the greater the superiority of the tufa cements is shown to be. Broadly speaking, sand briquettes containing 50% of tufa cement show marked superiority in ultimate strength over straight cement sand briquettes.

TABLE 5.—TUFA SAND BRIQUETTE TESTS, WITH VARYING PROPORTIONS OF TUFA.

Briquette number.	Date made.	Percentage of water.	Tensile Strength. 3 days.	7 days.	28 days.	3 months.	6 months.	Brand.	Percentage of tufa.	Fineness. 100 M.	200 M.	Setting Time. Initial. H.	M.	Final. H.	M.	Boiling test, 6 hours.
21	1911 Nov. 28	9½	85	200	335	370	330	Monolith cement.	55	....	91	2	00	4	20	O. K
			90	210	345	370	395	Monolith tufa.								
22	Dec. 11	9½	30	100	300	460	390	"	60	....	92	2	10	5	00	"
			35	100	310	435	385									
23	11	9½	35	90	250	350	330	"	70	....	90	2	00	4	00	"
			35	100	260	355	365									
24	11	9½	15	75	210	300	325	"	75	....	90	2	10	5	10	"
			25	80	200	285	375									
25	11	9½	15	75	150	255	300	"	80	....	90	2	50	5	00	"
			20	80	160	235	305									
26	11	9½	15	70	90	140	230	"	85	....	90	3	00	6	10	"
			20	70	90	110	260									

Table 5 shows a series of tufa cement laboratory tests with varying proportions of Monolith tufa with Monolith cement. This table is not in harmony with the mill-run tests shown in Fig. 1, but it appears to be the more rational. It will be noted that there is far less difference in the strengths at the end of 3 months than at the end of 7 days. Unfortunately, the results of the 1-year tests are not yet available.

Table 6 shows the tests of a mill run of 75% Haiwee tufa. It will be noted that the 6-month tests show a fine increase in strength over the 28-day tests.

TABLE 6.—SAND BRIQUETTE TESTS OF 75% MONOLITH-HAIWEE TUFA.

Briquette number.	Date made.	Percentage of water.	Tensile Strength.				Brand.	Percentage of tufa.	Fineness.		Setting Time.				Boiling test, 6 hours.
			3 days.	7 days.	28 days.	3 months.			100 M.	200 M.	Initial.		Final.		
											H.	M.	H.	M.	
15 Tests	1911 Nov.	....	52.8	86	201.6	365	Monolith Haiwee tufa	75	99	91	2	00	5	35	O. K.

Fig. 2 shows graphically the average of all breaks of three tufa cements manufactured by the City of Los Angeles during September, 1911. The standard Monolith cement is blended with 50% of tufa by volume. This is the standard practice on the aqueduct work. In making the test briquettes, the straight cement and tufa cement are mixed with standard sand in the ratio of 3 to 1 by weight, except in the one case shown, where the mix is 3 to 1 by volume. The tufa cement weighs 83 lb. per cu. ft., and the straight cement 95 lb. Standard sand weighs 110 lb. per cu. ft. A mixture by weight, as compared with volume, between tufa cement and straight cement, therefore, gives the former an advantage of 14% in the quantity of cement used. However, in mixing the tufa cement with sand by volume, and straight cement with sand by weight, and making the comparison, this is reversed, as the sand weighs 16% more than straight cement. In making concrete the field practice is to mix by volume, which, in the briquette, gives an idea of the strengths obtained in field practice with the tufa cements. Fig. 2 shows that the tufa cements are slower in getting their strength, usually attaining equal strength with the standard cement in from 6 to 10 days, but continuing to grow in strength, as far as observed, as shown in Fig. 3. All tests made in the aqueduct laboratories indicate this continued growth in strength of tufa cements. The standard cement, however, shows a loss in strength between 1 and 4 months, and then a slow recovery. Other California cement tested in the aqueduct laboratories shows similar loss.

In addition to the laboratory tests of the strength of the tufa, the sands and gravels which are used along the line of the aqueduct, were made into concrete with the tufa cements, and cast into test

slabs, 6 ft. wide and 12 ft. 5½ in. long, and loaded to destruction. These slabs were similar to those used in covering the aqueduct. Table 7 shows the details of these tests. The slabs were reinforced as indicated, and in a manner similar to the reinforcement used in the construction of the aqueduct. The wire mesh used is manufactured by

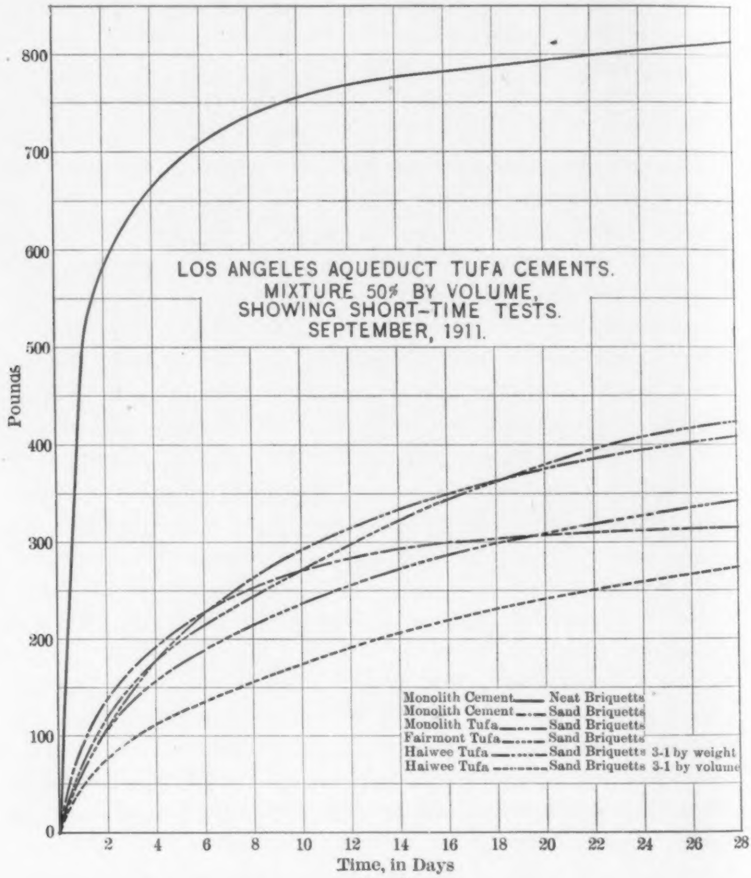

FIG. 2.

the American Steel and Wire Company, being from 58 to 42 in. wide. The practice was to roll these bundles longitudinally along the roof of the aqueduct. The mesh is triangular, 4 in. on a side, with No. 12 longitudinal and No. 14 diagonal wires. The concrete was made by hand, covered with a layer of earth and kept wet for 20 days, after

which the slabs were dried until they were tested. The slabs were put over piers with a clear span of 11 ft. 5⅜ in. A water load was used, a large canvas bag being put inside of a wooden frame. Tests were first made with earth loads, but the arching effect of the earth destroyed their value. Tufa cements of varying proportions were used, and, in these field tests, the 50% blend gave the most satisfactory results. The tufa cement concretes were stronger than the others. The tests with the straight cement slabs, unfortunately, were made with the reinforcing rods running straight across the bottom of the slab, instead of being bent up at the two sides, as was the case with the tufa cement slabs. In no case were the rods broken in the tests.

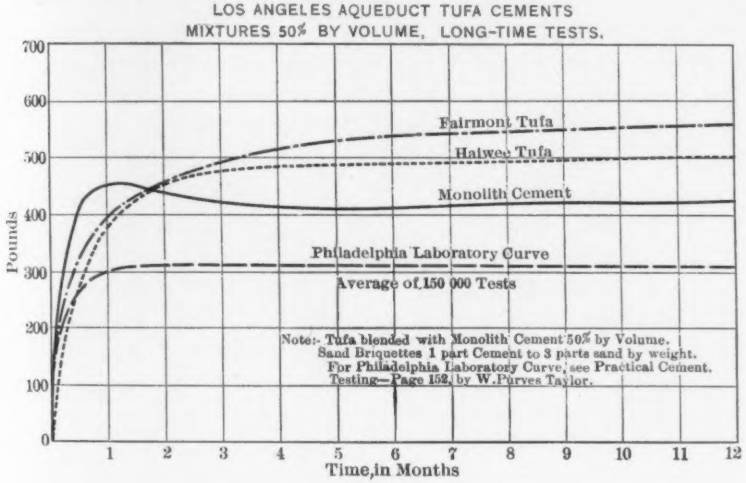

FIG. 3.

The tufa concrete had greater flexibility than the straight cement concrete. After having made this series of tests, the tufa cement was adopted for all classes of construction work on the aqueduct, including the concrete pipe. Five concrete pipes, 10 ft. in diameter, and having a 9-in. shell, reinforced with circular rods 4 in. apart, have now been made of tufa cement, the mixtures being 1 : 2 : 4. These pipes have been made for heads up to 75 ft. They have all been filled with water but one, which is necessarily empty until certain other connecting work is completed. Where they have been tested, the pipes are tight, with one exception where a slight circular crack developed. When this pipe was filled with water and soaked up, the crack closed

TABLE 7.—TESTS OF CONCRETE SLABS MADE OF TUFA CEMENT.

Slab no.	Kind of cement.	Proportions of mix.	Date made.	Age.	Description. Dimensions of slabs. Reinforcement: kind, size, how placed.	Manner of loading.	Depth of equivalent earth load at 100 lb. per cu. ft.	Total loads.	Deflections.	Remarks.
3	Tufa-Portland, 50% tufa.	1 : $2\frac{1}{2}$: 5	2/28	30	12 ft. $5\frac{1}{2}$ in. by 6 ft. 0 in. 8 in. thick at center. 6 in. thick at ends. Four $\frac{5}{8}$-in. twisted rods, spaced 18 in. center to center: $\frac{1}{2}$ in. from bottom of slab at center: bent to middle at ends. Wire mesh.	Water load	0 ft. $8\frac{1}{4}$ in.	5 000	$\frac{1}{16}$ in.	Crack at 6 000 lb
							1 " $4\frac{1}{2}$ "	10 000	$\frac{1}{4}$ "	
							2 " 1 "	15 000	$\frac{1}{2}$ "	
							2 " 9 "	20 000	$\frac{11}{16}$ "	
							3 " $5\frac{1}{2}$ "	25 000	1 "	
							4 " $1\frac{3}{4}$ "	30 000	$1\frac{5}{16}$ "	Poor bearing-R. Broke within 8 in. of rear end.
							4 " 4 "	32 225		
4	Tufa-Portland, 40% tufa.	1 : $2\frac{1}{2}$: 5	2/24	30	Ditto.	Ditto.	0 " $8\frac{1}{4}$ "	5 000	$\frac{1}{8}$ "	Crack at 1 000 lb.
							1 " $4\frac{1}{2}$ "	10 000	$\frac{7}{16}$ "	
							2 " 1 "	15 000	$\frac{15}{16}$ "	
							2 " 9 "	20 000	$2\frac{1}{16}$ "	
							2 " $10\frac{1}{2}$ "	21 000		Failure. Broke in center. Slow break.
5	Tufa-Portland, $62\frac{1}{2}$% tufa.	1 : $2\frac{1}{2}$: 5	3/9	30	Ditto.	Ditto.	0 " $8\frac{1}{4}$ "	5 000	$\frac{1}{4}$ "	
							1 " $4\frac{1}{2}$ "	10 000	$\frac{7}{16}$ "	
							2 " 1 "	15 000	$\frac{13}{16}$ "	
							2 " 9 "	20 000	$1\frac{5}{16}$ "	
							3 " 4 "	24 000	$2\frac{3}{16}$ "	
							3 " $5\frac{1}{2}$ "	25 000		Broke left end. Slow break.
6	Monolith Portland.......	1 : $2\frac{1}{2}$: 5	2/10	30	12 ft. $5\frac{1}{2}$ in. by 6 ft. 0 in. 6 in. uniform thickness. Four $\frac{5}{8}$-in. square twisted rods, $\frac{1}{2}$ in. from bottom of slab. Straight rods. Wire mesh.	Ditto.	0 " $8\frac{1}{4}$ "	5 000	$\frac{3}{16}$ "	
							1 " $4\frac{1}{2}$ "	10 000	$\frac{9}{16}$ "	
							2 " 1 "	15 000	$1\frac{7}{16}$ "	Broke 4 ft. from left end. One of the end rods pulled out.
							2 " $3\frac{1}{8}$ "	16 500		
1	Tufa-Portland, 30% tufa.	1 : $2\frac{1}{2}$: 5	2/12	31	12 ft. $5\frac{1}{2}$ in. by 6 ft. 0 in. 8 in. thick at center line. 6 in. thick at ends. Four $\frac{5}{8}$-in. square twisted rods, spaced 18 in. center to center: $\frac{1}{2}$ in. from bottom of slab at center; bent to middle at ends. Wire mesh.	Ditto.	0 " $8\frac{1}{4}$ "	5 000	$\frac{1}{8}$ "	
							1 " $4\frac{1}{2}$ "	10 000	$\frac{5}{16}$ "	Crack at 7 500 lb.
							1 " $8\frac{3}{4}$ "	12 500	$\frac{5}{8}$ "	
							2 " 1 "	15 000	$\frac{3}{4}$ "	
								15 050		Failed. Rods taken out.
2	Tufa-Portland, 50% tufa.	1 : $2\frac{1}{2}$: 5	2/17	....	Ditto.	Ditto.	0 " $8\frac{1}{4}$ "	5 000	$\frac{1}{16}$ "	Crack in center.
							1 " $4\frac{1}{2}$ "	10 000	$\frac{3}{16}$ "	
							1 " $8\frac{3}{4}$ "	12 500	$\frac{1}{4}$ "	Crack $\frac{1}{4}$ pt. to left.
							2 " 1 "	15 000	$\frac{3}{8}$ "	Crack at 14 000 quarter to right.
							2 " 5 "	17 500	$\frac{7}{16}$ "	
							2 " 9 "	20 000	$\frac{9}{16}$ "	
							3 " $5\frac{1}{2}$ "	25 000	$\frac{3}{4}$ "	
							4 " $1\frac{3}{4}$ "	30 000	1 "	
							4 " 10 "	35 000	$1\frac{5}{16}$ "	No failure. Test discontinued. Permanent set of $1\frac{3}{4}$ in. when load taken off.
							5 " $2\frac{1}{4}$ "	37 500	$1\frac{1}{2}$ "	
							5 " $6\frac{3}{4}$ "	40 000	$1\frac{7}{8}$ "	
							5 " $10\frac{5}{8}$ "	42 500	$2\frac{3}{16}$ "	

and became tight. In no case have any longitudinal cracks developed, and it is believed that the entire water load is carried by the concrete alone. When concrete takes its set, it shrinks slightly and throws the circular reinforcing steel under compression. Before the steel can carry the load, it must come under tension, and experience with concrete pipe elsewhere indicates that there is enough movement between these two conditions of tension and compression of the steel to cause a longitudinal crack in the concrete unless it carries the entire load. As a practical working test, therefore, this pipe of tufa concrete demonstrates the quality of the material.

The tufa concrete has also been used successfully in the lining of some tunnels in which the ground is exceedingly heavy, and where the sets in the tunnel, made of 10 by 10-in. timbers, spaced from 2 to 3 ft. apart, were repeatedly crushed. The tunnel lining, which has a theoretical thickness between posts of 14 in., has not shown any failure. In the heaviest ground, however, 6-in. steel I-beams were placed and wedged up against the lagging, the wooden sets then being taken out. This was done because in some instances the timbers were so close together that they reduced seriously the thickness of the concrete tunnel walls. The steel I-beams are left embedded in the concrete as a reinforcement.

Laboratory tests of the tufa product are made continuously at all the tufa mills, and samples are also sent to the main laboratory at the Monolith cement plant. Figs. 2 and 3 represent an average of a month's breaks of briquettes made with equal parts by volume of straight cement and tufa mixed with three parts of standard sand. These are fairly typical of the average monthly mill runs, the tufa cements showing better results than the straight cements and also showing a continued hardening. In the case of the German trass cements, this hardening is known to continue for a period of five years. The lower line on Fig. 3 shows the average of a great number of breaks of various Portland cements at the Philadelphia Testing Laboratories.

A striking feature of the tufa cement is that, in all the four years in which it has been tested, there has never been a pat which failed under the boiling test. This indicates, further, that any free lime which may occur in the cement combines with the silicas in the tufa. Microscopic slides were made of some of the tufa cement and

sand briquettes. They were examined, but no satisfactory conclusion could be reached.

Samples of tufa cement were sent to the Bureau of Standards of the United States Department of Commerce and Labor, and were tested in the Pittsburgh Laboratory. The following quotation is from a letter from this Bureau under date of March 29th, 1911:

"* * * You desired particularly to know whether there was any chemical reaction between the tufa and the cement. The enclosed report shows that such has undoubtedly been the case.

"The addition of tufa or puzzuolana to Portland cement undoubtedly does not reduce the strength of the latter when in the form of a mortar or concrete, but there has always been a question as to whether this is due to purely chemical or chemical and physical phenomena. There has not been any doubt that there is a reaction between the cement and the tufa, but there always has been a doubt as to whether this reaction was sufficient to account for the usual strengths developed by such mixtures.

"The tests were conducted as follows: Mixtures of two parts Riverside cement with one part of tufa, and two parts Riverside cement with one part Ottawa Standard sand, ground 90% through a 200-mesh sieve, were made into briquettes and broken at the end of one-week, four-week, and thirteen-week periods. At the same time, a similar series, using Atlas cement, was carried on, and also the same mixtures used in connection with 1:3 sand briquettes. After breaking the thirteen-week period briquettes, they were dried, the outside surface completely removed, and the interior ground to pass 200 mesh, and used for determinations. By determining the amount of insoluble silica in the cements and in the tufa and quartz, it is possible to calculate the amount of insoluble silica which should be in the dry briquettes (by dry, meaning the complete expulsion of water and CO_2 at 1 000 degrees C). From the analyses of these residues, there was also obtained the insoluble silica actually present. The difference between these two gives the amount of silica rendered soluble during this period of thirteen weeks. The figures are given in the following table:*

"The following results were obtained in breaking the briquettes:

Riverside.	2 Riverside, 1 tufa.	2 Riverside, 1 quartz.	
0.53	30.97	34.02	SiO_2 insoluble, calculated.
0.53	23.80	32.76	SiO_2 insoluble, determined.
	7.17	1.26	SiO_2 rendered soluble.

* "Tests made at Monolith show 8.4% of Monolith tufa rendered soluble; of the Italian Puzzuolana, 1.6%, and of Italian Tufa, 2 per cent. The presence of this so-called 'soluble silica' is what makes tufas preferable to sands for blending with Portland cements."

Atlas.	2 Atlas, 1 tufa.	2 Atlas, 1 quartz.	
0.59	30.80	33.89	SiO_2 insoluble, calculated.
0.59	24.01	33.00	SiO_2 insoluble, determined.
	6.71	0.89	SiO_2 rendered soluble.

TABLE 8.—Los Angeles Aqueduct Tufa Compared With Silica Cement.

			1 week.	4 weeks.	13 weeks.
Sand.	Riverside cement	1	307	370	416
		2	303	383	413
		3	310	394	390
	Average		307	382	403
	2 Riverside cement 1 Tufa, 90% through 200 mesh	1	215	373	458
		2	211	302	585
		3	221	341	438
	Average		215	339	457
	2 Riverside cement 1 Quartz sand, 90% through 200 mesh	1	210	272	286
		2	190	286	286
		3	188	281	303
	Average		196	280	292
	Tufa cement, 50% by volume Tufa and Riverside cement	1	262	488	534
		2	269	476	561
		3	235	466	478
	Average		255	477	524
Sand.	Atlas cement	1	357	341	454
		2	299	351	484
		3	281	335	469
	Average		312	342	466
	2 Atlas cement 1 Tufa, 90% through 200 mesh	1	212	350	462
		2	212	398	471
		3	200	410	...
	Average		208	386	466
	2 Atlas cement 1 Quartz sand, 90% through 200 mesh	1	205	265	314
		2	200	283	362
		3	200	255	367
	Average		203	268	348

The neat briquettes made with tufa cement by the Bureau do not show as much strength as those made with straight cement. This is in harmony with tests made in the aqueduct laboratories. However, when cement is used in concrete, it is always blended with sand, and the sand briquettes tested correspond to practice in construction work. The tests made with the sand briquettes show that the tufa cements have as good or better strength than the straight cements, and that the blend made of equal parts of tufa and cement is stronger than that made with but 33% of tufa. They also show clearly that the tufa cement, when mixed with sand, gives much better strength than the silica cement, indicating that the tufa combines in a different manner from the silica. In the neat briquettes, however, the silica cements develop the greater strength. The characteristic of the tufa cement continuing to harden substantially with age is indicated in these sand briquette tests. At the time the samples were sent to Pittsburgh, the Fairmont tufa was being blended with Riverside Portland cement. All other tests given of Los Angeles aqueduct tufas were made with the city's cement known as Monolith.

A review of an elaborate series of tests of the German trass cement, is given in a recent engineering periodical.* These are the best available laboratory tests of tufa cements. The investigations were carried on especially to determine the effect of sea water on cements of this class. They were made under the direction of the Prussian Minister of Public Works, and a committee consisting of representatives of the Royal Testing Laboratories, of the cement and trass industries, and Dr. Michaelis, cement specialist. The Royal Testing Laboratories were placed in their charge. Cements of two classes were used, those rich in lime and those poor in lime, and also mortars to which had been added trass and finely ground quartz sand, in order to determine whether trass only acts mechanically by increasing the density of the mortar, or chemically also. The tufas blend slightly better with the cements which are richer in lime. The results of these tests show that the addition of certain puzzuolana (tufa) materials to lean cement mortar is valuable in sea water. The detailed table (Table 9) is given because of the variety in the record.

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* *Engineering Record*, August 27th, 1910.

the long period of the tests, and the excellence of the authority. The tests run over a period of five years.*

This cement (Table 9) contained 65.80% of lime, and 23.74% of silica. The cements which have been blended with trass give as much tensile strength as the straight cement when made in sand briquettes, and the samples put in sea water show results of slightly less strength than when they are put in fresh water. The compressive strength of the concrete is less with the trass cements than with the straight cements.

An additional table† shows the strength of mortars and concrete made with a mixture of three parts of trass, two parts of hydrated lime, and one part of sand, giving tensile strengths of 216 lb. in 28 days, 356 lb. in 1 year, and 400 lb. in 5 years. A mixture made of 1½ of trass, 1 of hydrated lime, and 1 of sand gave a strength of 244 lb. in 28 days, 383 lb. in 1 year, and 360 lb. in 5 years.

It is noteworthy in Table 9 that the straight cement sand briquettes, in three out of four instances, show marked loss in tensile strength between 1 and 5 years, the first test alone showing constant strength, while the tufa (trass) combinations show gains in six out of eight cases during the same periods. The two tests showing loss are of samples in sea water.

Dr. W. Michaelis‡ has written an interesting paper on "Portland Cement Reground with Oregon Puzzuolana," in which he enters into a discussion of the chemistry of the problem, and makes a demonstration of the solubility of the tufa with the excess lime of the cement. He shows that puzzuolana (or tufa) will combine with hydrated lime. The series of tests given show the same general results as the tests with German trass cements (Table 9)—that, especially with the leaner mixtures, the tufa blends of equal parts are fully as strong in tension, or superior to, the straight cement mortars, and markedly better than the "silica cement." In the tests for compression in the leaner concretes (1:3:6), his tufas are as strong; but with a richer mixture (1:2:4), they are about 20% less strong than concrete made of straight cement with the same aggregates.

* The results have appeared in a report of the Royal Testing Laboratory under the title, "Mitteilungen aus dem Kgl. Materialprüfungsamt."

† *Engineering Record*, August 27th, 1910.

‡ *Cement and Engineering News*, November, 1911.

TABLE 9.—TESTS OF GERMAN TRASS CEMENTS IN SEA WATER (IN POUNDS PER SQUARE INCH).

	Stored in Fresh Water.						Stored in Sea Water.					
	Stored in sand 9 days.			Stored in sand 1 year.			Stored in sand 9 days.			Stored in sand 1 year.		
Age of samples.	Tensile strength.	Compressive strength. Mortar cubes.	Compressive strength. Concrete cubes.	Tensile strength.	Compressive strength. Mortar cubes.	Compressive strength. Concrete cubes.	Tensile strength.	Compressive strength. Mortar cubes.	Compressive strength. Concrete cubes.	Tensile strength.	Compressive strength. Mortar cubes.	Compressive strength. Concrete cubes.
1 Cement, 2 Sand.												
28 days....	572	6 760	7 040	....	......	......	646	6 790	7 170	....	......	......
1 year....	722	7 740	7 540	710	8 870	7 500	750	7 470	7 400	....	......	......
5 years...	722	6 320	7 590	750	9 530	7 720	612	7 620	7 700	859	11 900	7 820
1 Cement, 4 Sand.												
28 days....	348	2 990	3 390	....	......	......	362	2 960	3 520	....	......	......
1 year....	426	3 400	3 640	524	4 890	4 440	457	3 660	3 260	....	......	......
5 years...	362	3 320	4 030	439	3 760	4 340	369	3 440	4 420	442	4 040	4 440
1 Cement, 1 Trass, 4 Sand.												
28 days....	504	3 470	4 290	....	......	......	539	3 030	4 670	....	......	......
1 year....	616	5 860	5 520	622	5 970	5 820	749	5 900	5 340	....	......	......
5 years...	722	6 300	5 570	697	6 040	5 740	583	6 560	5 260	749	6 440	6 070
1½ Cement, 1 Trass, 5 Sand.												
28 days....	573	5 170	4 870	....	......	......	597	5 400	4 890	....	......	......
1 year....	640	7 200	5 670	617	6 970	6 100	672	7 220	6 120	....	......	......
5 years...	724	7 230	6 680	716	6 690	6 090	663	7 570	6 160	744	7 140	6 130
1 Cement, 1 Trass, 8 Sand.												
28 days....	242	1 560	2 120	....	......	......	292	1 700	2 140	....	......	......
1 year....	353	2 420	2 920	404	3 400	3 240	360	2 690	2 820	....	......	......
5 years...	399	2 820	3 230	370	2 370	3 060	394	2 890	3 090	341	2 490	3 120
1½ Cement, 1 Trass, 10 Sand.												
28 days....	240	1 430	2 140	....	......	......	262	1 430	2 220	....	......	......
1 year....	320	2 240	2 790	347	2 840	3 260	324	2 100	2 760	....	......	......
5 years...	329	2 220	3 090	419	3 020	3 330	329	2 240	2 900	437	3 260	3 260

Dr. Michaelis gives the following explanation of the chemical reactions that occur when the tufa combines with the cement:

"These desired hydrates of silica, alumina and iron oxide are found in nature in the form of puzzuolanas, or tufas, volcanic products created by the action of water or steam upon basaltic or granitelike molten formations. They can be artificially obtained by running molten blast furnace slag into water. In both cases the original compounds of the basalt, granite or slag are completely decomposed into their constituents and, furthermore, transformed into comparatively loose material which can easily be crushed. The most valuable part of the various chemical ingredients to be found in a natural or artificial puzzuolana is the silica hydrate, so-called 'soluble' silica which, in distinction from quartz silica, powdered quartz, is soluble in a 10 per cent. solution of sodium carbonate. Such silica combines readily with calcium hydrate and forms an excellent hydraulic cement. To what extent the alumina hydrate and iron hydrate combine with calcium hydrate has not been definitely ascertained. However, from recent researches it appears that especially the alumina hydrate is able to combine with a very large percentage of hydrated lime."

Cost to Manufacture.—The average cost of blending 1 bbl. of straight cement so as to produce 2 bbl. of tufa cement with the small mills installed on the Los Angeles aqueduct is about 74 cents, distributed as follows:

	Cost per barrel of blend.
General expense—labor, live stock, etc.	$0.04
Electric power, at 1.85 cents per kw-hr.	0.105
Quarrying	0.025
Mill operations	0.20
Net milling cost	$0.37

The process of blending 1 bbl. of straight cement with an equal part by volume of tufa gives a resulting product of approximately 10% in volume in excess of 2 bbl. of tufa cement. For this reason, a little more than 1 cu. ft. of tufa cement is put in a sack. A sack of tufa cement weighs 83 lb. The cost of milling tufa cement will vary with the density of the rock. This cost of 37 cents per bbl. of tufa cement applies to all three of the tufa-grinding plants. The tufa at Monolith is denser and slower to grind; but, as this tufa mill is a part of a larger plant, the milling costs are no greater than at the other two places.

Action of Tufa Cement in Field Work.—Tufa cement is more sensitive and requires greater care in curing than straight cement, because it is slower in reaching its final hardness. As a rule, Los Angeles aqueduct tufa cements will show as great strength in 7 days as the straight cements, and after that period the tufa cement gains in strength faster than the straight cement. (See Fig. 2.) The tufa cement has to be kept wet longer in hot weather to attain full strength, and is subject to frost longer in cold weather. In slab work, where it is supported by forms, the forms should be left in two or three days longer with tufa cement than with straight cement. When the aqueduct roof slab (which has a span of 11 ft. 5 in.) is made of tufa cement, the forms are stripped in 6 days, in moderate weather. The particular places for which tufa cement is adapted is in massive work, foundations, and in wet places. It is not claimed that it is suitable for high, thin walls exposed to the dry air of arid regions. It may be, but this has not yet been demonstrated on this work.

Gaugings made in arid America show that the greater portion of irrigation water diverted in earthen canals is lost by seepage before it reaches the fields. A lean tufa cement containing 75% of tufa could be used for earthen canal linings, and would be fully as dense as concrete made with straight cement. It would have sufficient strength to stand up on 1:1 slopes, and it can be given as smooth a surface as ordinary concrete. A length of several hundred feet of open canal lining of this kind has been put in the open flood section of the Los Angeles aqueduct, with 75% tufa. The concrete does not show up as hard in the field as the 50% tufa concrete, but it is satisfactory. Tufa cement, in being more finely ground, adheres somewhat more to the forms than concrete made with straight cement, which is a slight disadvantage. In places along the aqueduct where 50% tufa concrete joins concrete made with straight cement, both being a year old or more, no difference can be detected in the quality of the concrete by picking into it. Plaster made of tufa cement is smoother, and the laborers, after they get used to it, prefer it to straight cement plaster.

Prejudice Against New Cement.—Some cement manufacturers take a stand against tufa cement for two reasons: because it is a cheaper product, with which they would have to compete; and because,

having established a business for a standard Portland cement, anything which might be considered an adulteration would possibly mitigate against the reputation of all cements. This active opposition has already been encountered among the cement manufacturers in Southern California, who opposed the proposed building ordinance of Los Angeles containing a provision permitting the use of tufa cements. If, however, a product can be furnished which is cheaper in cost and as good in quality, the consumer should have the benefit of it, and undoubtedly will ultimately derive this benefit.

Some foremen and superintendents are also prejudiced against the use of a new product. This is true generally in various branches of industry, and it applies to tufa cement. On the Los Angeles aqueduct, it was found that some of the foremen at first endeavored to avoid the use of tufa cement, but now, after the lapse of two or three years, and having had some practical experience with it, they are willing to accept either that or straight cement from the city mills without any hesitation for all classes of work.

Other Combinations.—Diatomaceous earths are found at various places along the Pacific Coast. Their analysis is similar to the tufas, which they resemble somewhat in appearance and in physical characteristics. A test made with a diatomaceous earth found near Santa Barbara gave the following results:

	Monolith cement. 1 to 3 sand.	½ Monolith and ½ diatomaceous earth.
3 days	100	210
7 days	200	300
28 days	310	370
3 months	330	620

The briquettes were made as provided for in specifications for testing cements. Only one set of tests was made, and this table is not given as conclusive evidence.

In the Hawaiian Islands, the volcanic rocks prevail. As far as the writer's knowledge of that country extends, there are no lime deposits suitable for the manufacture of cement. The black basaltic lava has been analyzed by the branch of the Department of Agriculture located on the islands, and the following contents determined:

SiO_2	TiO_2	Fe_2O_3	Mn_3O_4	K_2O	Na_2O	CaO	MgO	FeO	Al_2O_3
51.98	1.50	2.90	0.92	0.97	2.70	9.57	5.61	6.84	15.85

It is difficult to grind this lava in the mills. There is another form of volcanic material, locally called red clinker, which resembles somewhat in appearance a cement clinker. Experiments were made in grinding with Santa Cruz cement both the basaltic lava and the red clinker in a local tube mill. The results are given in Table 10. After making the first three tests, $1\frac{1}{2}\%$ of gypsum was added, which improved the strength of the material. All these tests were made with mixtures of one part of the cement indicated to three parts of standard sand. While the results did not seem to be as satisfactory as those obtained with the Southern California tufas, nevertheless, enough strength was developed to indicate the possibility of blending these lavas with cement in such a way as to result in an economy.

It will be noted that the chemical analysis of the lava quite closely resembles the analysis of tufa, and it is this resemblance of chemical properties which suggested the experiments with the lavas. There is a marked difference in their physical characteristics as compared with the tufas. They have not been comminuted by contact with water as the tufas have, a process which is considered important, if not essential.

Conclusions.—The following conclusions are drawn for tufa or puzzuolana cements:

1st.—The tufa, when finely ground with cement and used in concrete, combines both chemically and mechanically. Blends of 50%, when mixed with sand, give greater tensile strength after 10 days than straight cement mixed with the same proportion of sand. The leaner the mixture, the greater the relative superiority of the tufa cement. In compression, the tufa cement concrete is less strong (20%) in rich mixtures (1:2:4), and as strong in leaner mixtures (1:3:6).

2d.—Tufa cements, in tension, of blends from 30 to 80% show a continued growth in strength with age, as far as tested, up to 5 years, and in this respect are superior to straight cements which usually show declining strengths.

3d.—The tufa concretes must be handled with greater care with reference to both cold and drying, and forms should be left in place about one-third longer. In massive work this feature is negligible.

4th.—From the fact that the tufa cement is more finely ground and, in part, combines mechanically with other aggregates, carrying

the gradation of fineness one step farther, it makes a denser and more impervious concrete.

5th.—Where cements are high priced, a substantial economy may be effected if deposits of tufa are available. These conditions occur in portions of Western America. The milling cost of producing the extra barrel of tufa cement in small plants should not exceed 75 cents.

TABLE 10.—Sand Briquette Tests Made with Hawaiian Lavas.

Briquette number.	Date made.	Percentage of water.	Tensile Strength.				Brand.	Percentage of tufa.	Fineness.		Setting Time.				Boiling test, 6 hours.
			3 days.	7 days.	28 days.	3 months.			100 M.	200 M.	Initial.		Final.		
											H.	M.	H.	M.	
a	1911 Nov.	10½	145 135	250 260	335 345	435 465	Santa Cruz cement and sand..........	....	93.8	......	2	15	6	30	O. K.
b	"	10½	55 105	165 175	250 245	320 330	Red clinker and sand.	50%	......	91	0	40	1	25	"
c	"	10½	110 75	140 150	185 195	250 275	Lava basalt..........	50%	......	92	0	40	1	20	"
27	Dec.	10½	35 35	100 110	145 150	....	Red clinker Santa Cruz..............	50%	......	91.4	0	40	1	25	"
28	"	10½	125 130	200 210	325 355	....	Red clinker, 1½% gypsum added...	50%	......	......	2	30	6	00	"
29	"	10½	100 110	200 200	340 350	....	Red clinker, 1½% gypsum added...	50%	......	......	2	30	6	00	"
30	"	10½	125 130	190 200	330 335	....	Red clinker, 1½% gypsum added...	50%	......	......	2	30	6	00	"
31	"	10½	145 145	190 200	310 325	....	Red clinker, 1½% gypsum added...	50%	......	......	2	30	6	00	"
32	"	10½	120 120	215 230	350 340	....	Red clinker, 1½% gypsum added...	50%	......	......	2	30	6	00	"
33	"	10½	135 140	200 200	320 365	....	Red clinker, 1½% gypsum added...	50%	......	......	2	30	6	00	"
34	"	10½	120 140	195 200	350 345	....	Red clinker, 1½% gypsum added...	50%	......	......	2	30	6	00	"
35	1912 Jan.	....	145 140	245 255	330 330	....	Red clinker, 1½% gypsum added...	50%	......	......	2	30	6	00	"
36	"	....	130 140	220 225	300 310	....	Red clinker, 1½% gypsum added...	50%	.....	......	2	30	6	00	"
37	"	10½	55 55	120 125	180 190	....	Blue lava.......	50%	......	......	1	00	11	00	"
38	"	10½	80 80	135 150	275 275	....	"	50%	......	......	1	00	11	00	"
39	"	10½	60 65	150 160	240 255	....	"	50%	......	......	1	00	11	00	"

The development of the tufa cement on the aqueduct, as is usually the case with affairs of this kind, is the result of the co-operation of a number of different parties. Mr. E. Duryee, Cement Chemist for

the aqueduct at that time, conducted the preliminary experiments. Mr. G. M. Andrews, who succeeded Mr. Duryee as Cement Chemist, has done a great deal in investigating these cements. The cement plant is under the direction of Mr. Roderick MacKay, Mechanical Constructor, and William Mulholland, M. Am. Soc. C. E., is Chief Engineer in general charge of the work on the aqueduct. The writer has been Assistant Chief Engineer since the beginning of the work.

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PAPERS AND DISCUSSIONS

A SHORTENED METHOD IN ARCH COMPUTATION.*

By H. A. Sewell, Esq.†

In the design of elastic arches, as given by William Cain, M. Am. Soc. C. E., the method of single loads is more accurate than that of resultant moment polygon, because it computes the moment and thrust for each load separately, while the latter computes these quantities for all the loads together. Thus, in computing

$$\Sigma\,(mb'\,y) = \frac{\Sigma\,(bh)\,\Sigma\,(y)}{N} - \Sigma(bh'\,y),$$

the two quantities in the right member of the equality are so nearly equal in the latter method that they must be multiplied out by long hand, hence multiplying errors; while in the former the quantities dealt with are so much smaller that an ordinary slide-rule will usually suffice, thus eliminating false accuracy.

On the other hand the polygon method requires the computation of, at most, only six polygons, corresponding to different positions of the live load; while the method of single loads requires as many polygons as there are loads to the left of the crown, although these latter are somewhat easier to compute.

Because of its greater accuracy, and because it determines the exact position of the live load for maximum moment at any given section, rather than assumes its arbitrary position for maximum moments, the single-load method, doubtless, would be much more widely used,

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* This paper will not be presented at any meeting, but written communications on the subject are invited for publication with it in *Transactions*.

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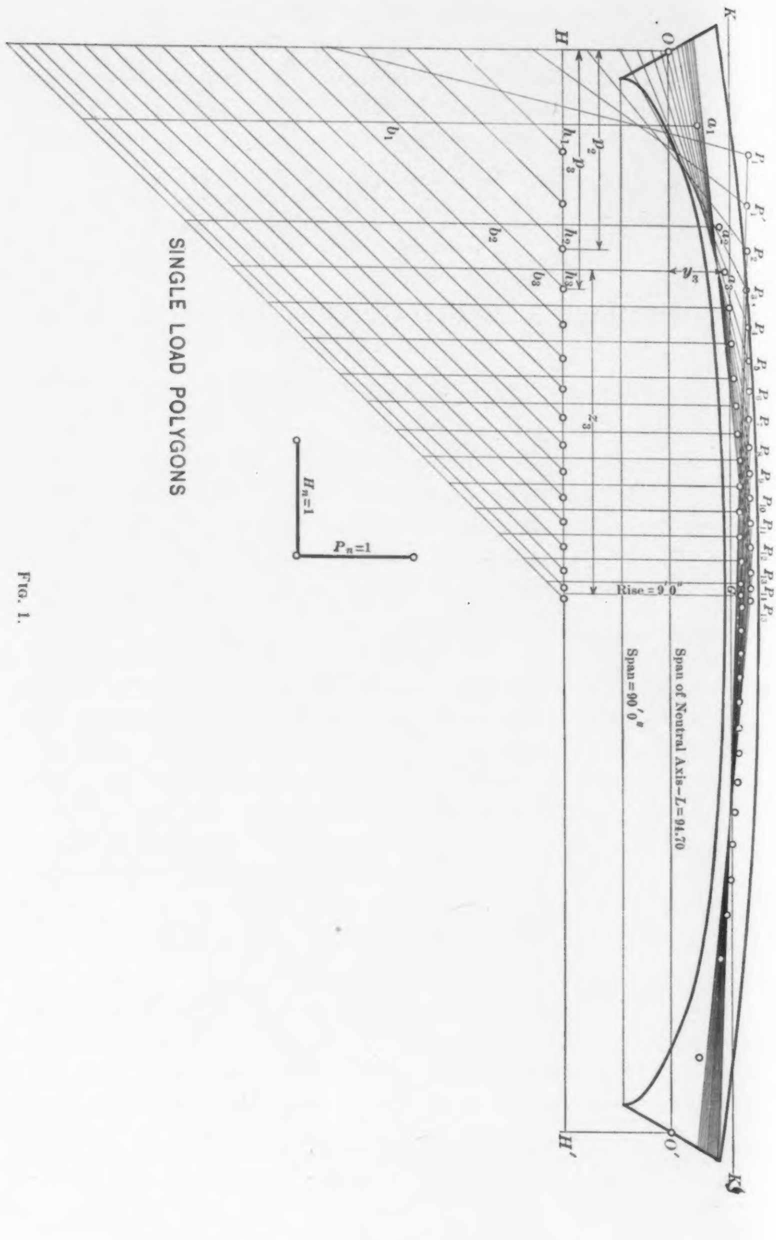
SINGLE LOAD POLYGONS
Span of Neutral Axis–L= 94.70
Span=90′ 0″
Rise = 9′ 0″
H_n=1
P_n=1

FIG. 1.

TABLE 1.

Point	(z)	(y)	(z^2)	(y^2)	Load	p	$\frac{L}{2}-p$	$p_n - p_{(n-1)}$	$\Sigma_o^{n-1}(z)$	$\Sigma_o^{n-1}(y)$	$\Sigma(bh)$	$\Sigma(bh'z)$	$\Sigma(bh'y)$
a_1	40.80	2.50	1664.0	6.25	P_1 P_1'	8.90 13.42	38.45 33.93	8.90 4.52			2.35 6.87	96.0 280.5	5.88 17.18
a_2	31.97	4.43	1022.0	19.62	P_2	17.35	30.00	3.93	40.80 31.97	2.50 4.43	3.93 1.97 12.77	160.5 63.0 504.0	9.83 8.73 35.74
a_3	28.13	4.95	791.5	24.50	P_3	20.88	26.47	3.53	72.77 28.13	6.93 4.95	7.06 1.66 21.49	257.0 46.7 807.7	24.46 8.22 68.42
a_4	24.84	5.25	617.0	27.55	P_4	24.04	23.31	3.16	100.90 24.84	11.88 5.25	9.48 1.53 32.50	318.6 38.0 1164.3	37.52 8.03 113.97
a_5	21.82	5.52	496.0	30.50	P_5	26.96	20.39	2.92	125.74 21.82	17.13 5.52	11.68 1.43 45.61	367.0 31.2 1562.5	50.05 7.90 171.92
a_6	19.02	5.75	361.8	33.05	P_6	29.64	17.71	2.68	147.56 19.02	22.65 5.75	13.40 1.31 60.32	395.3 24.9 1982.7	60.70 7.53 240.15
a_7	16.47	5.92	271.2	35.05	P_7	32.13	15.22	2.49	166.58 16.47	28.40 5.92	14.94 1.25 76.51	415.0 20.6 2418.3	70.80 7.41 318.36
a_8	14.00	6.05	196.0	36.60	P_8	34.54	12.81	2.41	183.05 14.00	34.32 6.05	16.87 1.19 94.57	441.5 16.7 2876.5	82.78 7.20 408.34
a_9	11.67	6.16	136.0	37.95	P_9	36.83	10.52	2.29	197.05 11.67	40.37 6.16	18.32 1.15 114.04	451.3 13.3 3341.1	92.45 7.09 507.88
a_{10}	9.42	6.25	88.7	39.10	P_{10}	39.04	8.31	2.21	208.72 9.42	46.53 6.25	19.89 1.11 135.04	461.5 10.5 3813.1	102.95 6.95 617.77
a_{11}	7.25	6.32	52.6	39.95	P_{11}	41.18	6.17	2.14	218.14 7.25	52.78 6.32	21.40 1.08 157.52	467.0 7.8 4287.9	113.00 6.83 737.60
a_{12}	5.14	6.37	26.4	40.60	P_{12}	43.27	4.08	2.09	225.39 5.14	59.10 6.37	22.99 1.06 181.57	471.0 5.4 4764.3	123.50 6.76 867.86
a_{13}	3.06	6.39	9.4	40.80	P_{13}	45.33	2.02	2.06	230.53 3.06	65.47 6.39	24.72 1.04 207.33	475.0 3.2 5242.5	134.80 6.65 1009.31
a_{14}	1.01	6.40	1.0	40.90	P_{14}	46.85	0.50	1.52	233.59 1.01	71.86 6.40	19.76 0.51 227.60	355.2 0.5 5598.2	109.20 3.26 1121.71
Σ_0^{14}	234.60	78.26	5713.6	452.42	P_{15}	47.85	−0.50	1.00	234.60	78.26	14.00 241.60	234.6 5832.8	

especially for very flat or very high arches, were it not for the much greater labor involved. However, in the following method, the computations are greatly reduced, while, at the same time, a check is introduced, which cuts the labor of computing moments again by making the computations self-checking.

The summations, $\Sigma\,(y)$, $\Sigma\,(y^2)$, $\Sigma\,(z)$, and $\Sigma\,(z^2)$, having been computed for the arch ring by the usual method, the quantities, $\Sigma\,(bh)$, $\Sigma\,(bh'\,z)$, and $\Sigma\,(bh'\,y)$, should next be obtained for each of the trial polygons corresponding to loads unity and horizontal thrusts unity at each of the load points, P_n.

The force polygon for load unity and horizontal thrust unity is taken so that the pole point is one unit horizontally to the left of the lower extremity of the load vector unity, thus making the right component of each trial-moment polygon horizontal and the left component inclined downward at an angle of 45 degrees. By reference to Fig. 1, the following relations are discovered:

$$\sum_o^n (bh) = \sum_o^{n-1} (bh) + (n-1)\,(p_n - p_{(n-1)}) + \left(p_n - \frac{L}{2} + z_n\right)$$

$$\sum_o^n (bh'\,z) = \sum_o^{n-1} (bh'\,z) + (p_n - p_{(n-1)}) \sum_o^{n-1} (z) + \left(p_n - \frac{L}{2} + z_n\right) z_n$$

$$\sum_o^n (bh'\,y) = \sum_o^{n-1} (bh'\,y) + (p_n - p_{(n-1)}) \sum_o^{n-1} (y) + \left(p_n - \frac{L}{2} + z_n\right) y_n$$

in which:

L = span of the neutral axis;
y = ordinate at a of arch from horizontal through spring, OO';
z = distance of a to left of crown, G;
p = distance of load considered from left spring, O;
n = number of load considered from left;
bh = ordinate of polygon from horizontal, HH'.

Thus, each summation is made to depend on the one preceding it and the quantities $(p_n - p_{(n-1)})$ and $\left(p_n - \frac{L}{2} + z_n\right)$. The work of these computations on a hypothetical flat arch is shown in Table 1.

In order to make the use of the formulas clear, a case is taken, $P_n = P_3$, from Table 1. Then for $P_{(n-1)} = P_2$, $\Sigma\ (bh) = 12.77$, $\Sigma\ (bh'\ z) = 504.0$, and $\Sigma\ (bh'\ y) = 35.74$; and, by adding the values of $z_{(n-1)} = z_2$, and $y_{(n-1)} = y_2$ to the last totals, we obtain $\sum_o^{n-1} (z) = 72.77$ and $\sum_o^{n-1} (y) = 6.93$. Multiply these latter quantities by the value of $(p_n - p_{(n-1)}) = 3.53$ and place the products under the values of $\Sigma\ (bh'\ z)$ and $\Sigma\ (bh'\ y)$ given above; and multiply $(p_n - p_{(n-1)})$ by $(n - 1) = 2$, placing the result under $\Sigma\ (bh)$. Next obtain $\left(p_n - \frac{L}{2} + z_n\right) = z_n - \left(\frac{L}{2} - p_n\right) = 1.66$, and place it in the $\Sigma\ (bh)$ column; then multiply it by $z_n = 28.13$ and $y_n = 4.95$, placing the results in the $\Sigma\ (bh'\ z)$ and $\Sigma\ (bh'\ y)$ columns. Finally, add the quantities below the last addition, in order to obtain the totals, $\Sigma\ (bh) = 21.49$, $\Sigma\ (bh'\ z) = 807.7$, and $\Sigma\ (bh'\ y) = 68.42$.

The check on the totals is shown by $\Sigma\ (z)$ being equally distant from $\Sigma\ (bh)$ for the loads, P_{14} and P_{15}. Likewise, $\Sigma\ (z^2)$ should be midway between $\Sigma\ (bh'\ z)$ for the loads P_{14} and P_{15}. No check was discovered for $\Sigma\ (bh'\ y)$ except to compute $\Sigma\ (bh'\ y)$ for Point P_{14} independently by the usual method. $\Sigma\ (z)$ and $\Sigma\ (y)$ are checked by direct addition. Checking the totals for P_{14}, checks all the others, because they are all carried forward in making the totals. All the multiplications may be made with an ordinary slide-rule.

These summations being obtained, the work is carried forward in the usual manner as outlined by Professor Cain.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

THE ECONOMIC ASPECT OF SEEPAGE AND OTHER LOSSES IN IRRIGATION SYSTEMS.*

By E. G. Hopson, M. Am. Soc. C. E.

To be Presented December 4th, 1912.

In a report to the Comptroller of New York City, made by John R. Freeman, M. Am. Soc. C. E., in 1900, on the New York water supply, attention was drawn in a very clear and forceful manner to the enormous proportion of waste incident to the operation of a great city water-works system. The subject had been dealt with before by other engineers, and has been handled in a very comprehensive way by others since, but the writer did not recall at the time ever having seen the subject dealt with so comprehensively as in Mr. Freeman's report.

On page 38 of that report there is an interesting diagram showing the consumption of Croton water hour by hour during a typical week. By an ingenious interpretation of related, but more or less disjointed, bits of evidence, it was shown that of a daily delivery of 115 gal. to each inhabitant of the city, only about 40 gal. were really used and about 75 were wasted, that is, the proportion of use to waste was about 1:2.

It was further deduced that of the 75 gal. wasted, 65 was in all probability needless waste, and could be stopped by the adoption of proper measures. Naturally, the question arose as to whether it was worth while for a city to continue to lavish vast sums in the con-

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* Read at the Annual Convention of the Society, Seattle, Wash., June 27th, 1912.

struction of new works, the greater part of the product of which would flow into the sea without benefit to any one, or whether it would not be better policy to devote some of this money to internal improvements in works already built, whereby savings equivalent in their effect to extensions of the supply system could be effected. Since the Freeman report was made much additional information has been gained on waste and its prevention in city water-works systems, and it has been shown that the amount of what was termed by Mr. Freeman needless waste is not quite so great as has been supposed. The question as to whether enforced economy in use is better policy than increasing the capacity of the system is still, to a large extent, a debatable one.

The reasons in favor of moderate consumption and avoidance of waste apply with even greater force to an irrigation system than to a city water-works system, in spite of the fact that the cost of the latter is relatively much higher than that of irrigation works. In a great city the cost of water-works is a comparatively light burden to the community, the expense to the individual of an unrestricted supply of pure water being one of the smallest items in his annual expense account. On the other hand, anything in the nature of restriction in use directly affects the personal convenience of each inhabitant, and is resented; he often prefers paying an extra trifle in order to enjoy not only a sufficiency but an excess.

With an irrigation system conditions are different. Usually, the supply is limited in quantity, and a waste in one direction is immediately reflected by straitened conditions in another. A system of irrigation work is designed to supply a definite quantity of water to each acre of land. The engineer makes certain allowances for waste and losses by seepage and evaporation. If his calculations are correct, the land receives a supply considered by him as sufficient, but not excessive. If, however, through some unexpected cause, the waste or losses are greater than were anticipated, less land can be brought under cultivation than had been contemplated, or farmers are compelled to get along with less water than had been considered necessary; hence the results are felt immediately and directly.

In the case of irrigation, as with a water-works system, losses can be classed as curable and incurable, and it is the writer's purpose to consider briefly those classes, as illustrated by works constructed

FIG. 1.—COLD SPRINGS RESERVOIR, OREGON.

FIG. 2.—JOINTING 46-INCH CONCRETE PIPE, UMATILLA PROJECT.

by the Government in the Northwest during the last five or six years.

Roughly speaking, incurable losses in irrigation systems result directly through water lost by absorption in the beds of reservoirs and evaporated into the air. Curable loss lies almost wholly in that absorbed in the beds of canals and other conduits.

In the great storage reservoirs required for irrigation works it is obviously an economic impossibility to accomplish anything in the way of preventing absorption or seepage losses in their beds. Whatever losses result through this cause must be accepted as unavoidable. The engineer, however, must be prepared to accept responsibility for results, as his advice or decision on the all-important question of selecting or approving a reservoir site is the only safeguard against what may be disastrous loss if his judgment is ill-advised. For this reason the writer is illustrating the fundamental differences in conditions and results in four typical irrigation reservoirs built in the Northwest by the United States Government.

The East Park Reservoir is strictly a storage reservoir, built on a branch of Stony Creek, one of the Coast Range feeders of the Sacramento River. The dam site is a good one, being a notch in a great conglomerate dike or ridge that runs through the country in a north and south direction, and the dam is a solid masonry structure of the gravity type on an arched plan. The bed of the reservoir is practically wholly in the typical California shale. The dam was completed in 1910, and water was first stored in the winter and spring of 1910 and 1911. Weekly measurements are taken of the influent and effluent, the storage, and the rates of evaporation.

The maximum capacity of the reservoir is 45 000 acre-ft., and the maximum area of water surface is 1 690 acres. Table 1 shows the results in the season of 1910-11, the season being from November 1st to November 1st, in this and all the following cases.

TABLE 1.—EAST PARK RESERVOIR, 1910–11.

	Acre-feet.	Percentage of influent.
Influent	65 400	..
Effluent and losses:		
Evaporation	7 100	11
Use, waste and surplus	58 300	89
*Seepage	0	0

* No appreciable seepage loss.

This reservoir represents the highest condition of efficiency of any of the four described. The records fail to show any seepage loss, the only appreciable loss being that by evaporation; thus nearly 90% of the water entering this reservoir is available for use.

The Cold Springs Reservoir of the Umatilla Project, in Oregon, is a good average reservoir, from a Western standpoint. In the East it would probably not be regarded as a site of special promise. The dam is an earthen one, nearly 4 000 ft. long, of a maximum height of nearly 100 ft. The general structure of the country is volcanic, with vast overlying beds of stratified sands, gravels, and hardpan. The valley constituting the reservoir site is the outlet of some 200 sq. miles of drainage area with little or no ordinary run-off. The reservoir is supplied by a feed canal, some 25 miles long, diverting from the Umatilla River at times when the latter has available water. The capacity of the reservoir is 50 000 acre-ft., and its maximum area is 1 550 acres.

This reservoir was first placed in commission in the spring of 1908, and has been operated ever since. There are, therefore, four yearly records of results. In this case measurements were obtained with unusual accuracy, as practically all the inflow passed over a sharp-crested weir at the lower end of the feed canal, and the effluent was also carefully measured over another weir below the outlet gates. This reservoir shows losses ranging from 34 to 24% of the influent during the four-year period. Judging by the record of the past two years, it would appear that a fair condition of stability has been attained in the regimen, in which about one-fourth of the water entering this reservoir is subject to unavoidable loss through seepage and evaporation. Table 2 gives a summarized tabulation of the results.

The Clear Lake Reservoir, in California, situated just south of the California-Oregon line, is a feature of the Klamath Project. It occupies a great natural depression or sink, some 25 000 acres in extent, at the reservoir flow line. About one-half of the bed consists of a natural sink of alkaline water known as Clear Lake which for ages has received and evaporated the surplus waters of Willow Creek. This reservoir was built by the Government principally for the purpose of holding back the waters of Willow Creek, in order to facilitate the unwatering of lands marginal to Tule Lake, a body of water into which Willow Creek ultimately discharges. The reservoir was intended to

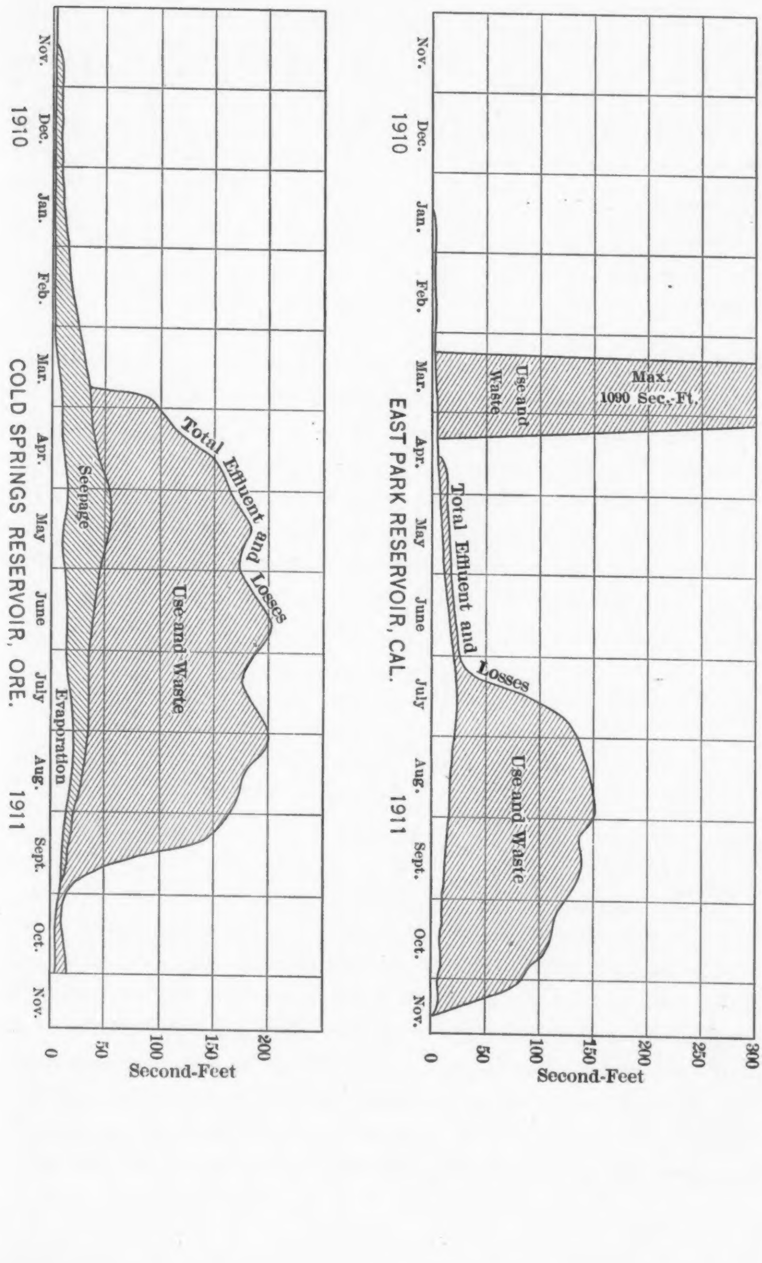

COLD SPRINGS RESERVOIR, ORE.
1910
1911
Nov.
Dec.
Jan.
Feb.
Mar.
Apr.
May
June
July
Aug.
Sept.
Oct.
Nov.
Total Effluent and Losses
Use and Waste
Seepage
Evaporation
0
50
100
150
200
Second-Feet
EAST PARK RESERVOIR, CAL.
1910
1911
Nov.
Dec.
Jan.
Feb.
Mar.
Apr.
May
June
July
Aug.
Sept.
Oct.
Nov.
Max. 1090 Sec.-Ft.
Use and Waste
Total Effluent and Losses
Use and Waste
0
50
100
150
200
250
300
Second-Feet

Fig. 1.

TABLE 2.—COLD SPRINGS RESERVOIR, OREGON.
Percentage of Losses Expressed in Terms of the Influent.

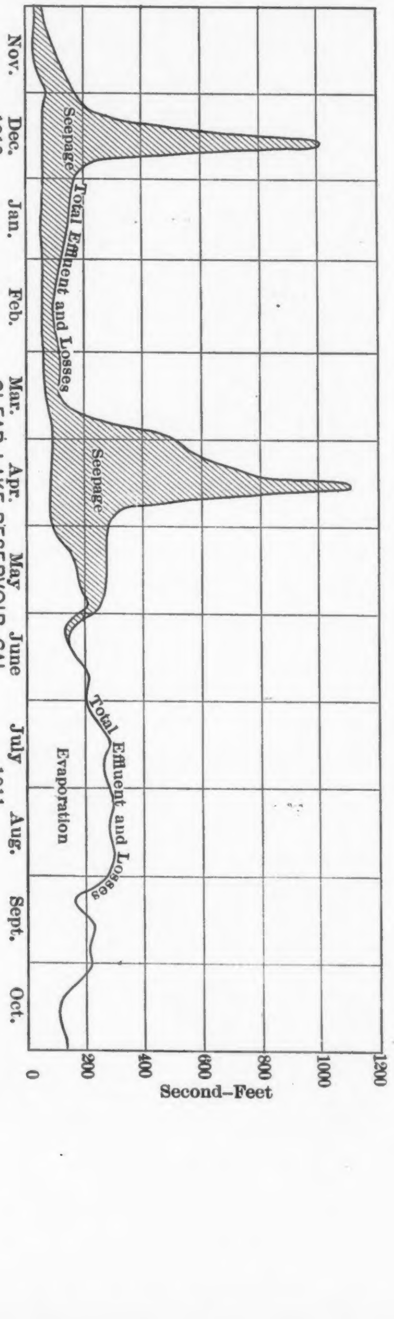
	1908.		1908-09.		1909-10.		1910-11.	
	Acre-feet.	Percentage.	Acre-feet.	Percentage.	Acre-feet.	Percentage.	Acre-feet.	Percentage.
Influent	20 366	..	42 820	..	61 526	..	72 273	..
Effluent and losses:								
Evaporation	2 400	12	4 295	10	5 333	9	6 252	9
Seepage	4 515	22	*4 021	9	*10 461	17	10 878	15
Use, waste, and surplus.	13 451	66	34 504	81	45 732	74	55 163	76
*Return flow	......	..	865	..	503	..	182	..

combine the purposes of a great evaporating pan and a regulator of the diversion channel that diverts the discharge of Lost River from Tule Lake into Klamath River. More recent plans, however, have considered its possibilities as a source of irrigation supply. The capacity of the reservoir is enormous as compared with the available water supply, being 450 000 acre-ft., with an area of 25 000 acres. The dam on Willow Creek is a rock fill structure some 30 ft. high, which was completed in 1909. There are two years' records of the action of this reservoir, as given in Table 3.

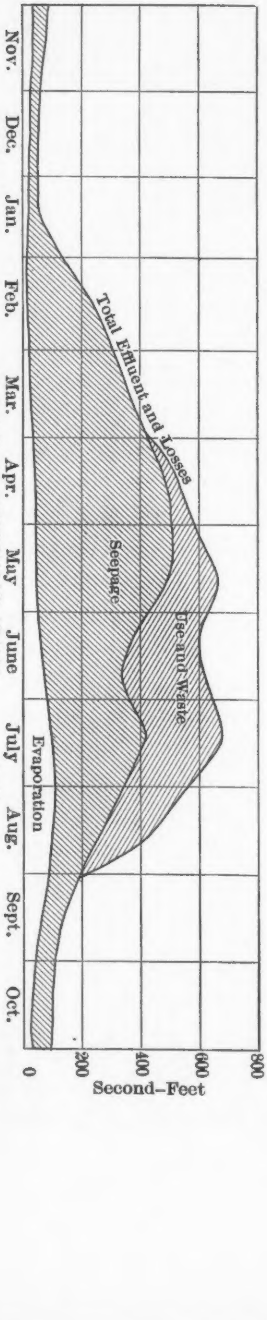
TABLE 3.—CLEAR LAKE RESERVOIR.

	1909-10.		1910-11.	
	Acre-feet.	Percentage.	Acre-feet.	Percentage.
Influent	141 000	....	225 000	....
Effluent and losses:				
Evaporation	80 000	57	88 000	39
Seepage	48 000	34	24 000	11
Use, waste, and surplus	13 000	9	113 000	50

The rate of evaporation in this vicinity has been estimated at a little more than 4 ft. in an average year. It will be noted that evaporation is the principal loss in the Clear Lake Reservoir, as had been anticipated. The seepage losses during the first year were heavy, but, apparently, the marginal lands have filled up so that the losses in 1911 were comparatively moderate. It is important to note that in a year of copious run-off, like 1910-11, as much as 50% of the



CLEAR LAKE RESERVOIR, CAL.



DEER FLAT RESERVOIR, IDAHO

FIG. 2.

supply was subject to unavoidable loss or waste, which, in this case, was intentional, the principal purpose of the reservoir being the disposition of surplus water, rather than its conservation for use.

The Deer Flat Reservoir, a feature of the Boise Project, in Idaho, presents different natural conditions from the three preceding types. It does not occupy a natural drainage valley or sink, but, on the contrary, is situated on a flat saddle between the hills, the lower ends of which are closed by two earthen dams. It has a maximum area at high-water line of 9 250 acres, with a maximum capacity of 186 000 acre-ft. The reservoir derives its supply, as in the case of the Cold Springs Reservoir, through a feeder canal, known as the New York Canal, diverting from the Boise River some 10 miles southeast of Boise. The reservoir was first placed in commission in 1909, and has been in operation ever since. The bed consists in large part of silts, sands, and gravels, with a covering of from 3 to 5 ft. of soil. Seepage losses in this case have been pronounced from the outset, and constitute the bulk of all losses. When the reservoir was first placed in commission almost 90% of the water entering it was lost by absorption in the reservoir bed. In that year, however, the reservoir was only filled to one-tenth of its capacity. During the next two seasons larger and larger quantities of water were introduced, and the proportion of losses has fallen appreciably, but still remains exceedingly high. During the last season about two-thirds of the water entering this reservoir was subject to loss through evaporation and seepage. It may be expected that conditions will improve at this point as the adjacent and underlying strata of the reservoir gradually become filled by the constant application of water, but the extent and period of these ameliorating conditions are quite uncertain. A summarized tabulation of results is given in Table 4.

TABLE 4.—DEER FLAT RESERVOIR.

	1909.		1909-10.		1910-11.	
	Acre-feet.	Percentage.	Acre-feet.	Percentage.	Acre-feet.	Percentage.
Influent	64 000	......	130 000	......	230 000	.....
Effluent:						
Evaporation	4 000	6	18 000	14	20 000	9
Seepage	55 000	86	80 000	62	140 000	61
Use, waste, and surplus	5 000	8	32 000	24	70 000	30

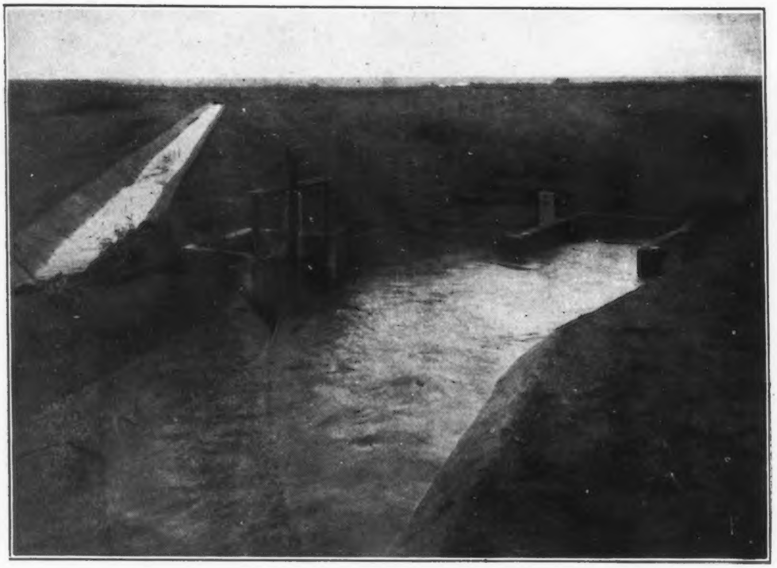


FIG. 1.—MORTAR-LINED LATERALS AND CONCRETE STRUCTURES, UMATILLA PROJECT.

FIG. 2.—MORTAR-LINED LATERAL, UMATILLA PROJECT.

The foregoing records, while incomplete and faulty in many respects, are among the best obtainable in a new country, and in any event are instructive. The general problem of reservoir losses is often given less attention by engineers than its importance warrants. In many cases the dam site is apt to monopolize attention, and an engineer accustomed to deal with reservoir sites in Eastern river valleys, where the adjacent water-tables are high and the losses are generally confined to evaporation, may be led to the commission of grave mistakes. A great deal has been said and written about return flow. One of the writer's earliest recollections in connection with reservoir studies was the discussions in the *Transactions* of the Society between Messrs. FitzGerald, Stearns, Fteley, and others, on groundwater storage of certain reservoirs in the East. Mr. FitzGerald's conclusions as to the general inadvisability of giving credit to the invisible storage of a reservoir are wise. Save under exceptional conditions, the writer doubts whether much, if any, additional draft can be made from Western reservoirs in excess of the visible storage. During the past four years the Cold Springs Reservoir has absorbed some 30 000 acre-ft. of water in its bed; it has apparently yielded back only about 1 500 acre-ft. The Deer Flat Reservoir has absorbed apparently 270 000 acre-ft., with little or no return.

It is important to note that in a reasonably good, representative, irrigation reservoir, such as Cold Springs, one-quarter of the water turned into it is lost, and that, apparently, under the most favorable circumstances, as at East Park, 10% will be lost.

The main lesson to be derived from these few illustrations is that the geologic structure of the site should be given the most careful consideration, as it is vital to determine in advance, as nearly as may be, the amount of reservoir losses, and whether they are likely to be of a permanent character.

On the Umatilla Project, the cost of the irrigation works per acre of irrigable land is from \$60 to \$70; on the Truckee-Carson Project, about \$40; on the Orland Project, about \$50; on the Tieton, about \$90; on the Sunnyside, about \$50; and on the Klamath, from \$30 to \$40; say, an average of about \$55. This is a fair indication of the general run of costs in large irrigation work in that part of the country, and is probably lower than the average costs on newer projects, either Government or private.

The various losses in the water-supply system, as expressed in percentages of water diverted, are as shown in Table 5.

TABLE 5.—PERCENTAGES OF LOSSES.

	Reservoir.	Canal losses.	Totals.
Umatilla	20	32	52
Truckee-Carson	0	41	41
Orland	8	23	31
Klamath	0	48	48
Tieton	0	24	24
Sunnyside	0	27	27

These losses, running from one-fourth to upwards of one-half of the whole supply, are, unfortunately, not all. They include only the losses from the diversions down to the end of the regular lateral systems operated by the Government; but below these are the ramifications of the small ditches built by the farmers to distribute water to their farms. These farm ditches are usually small earthen trenches, in which heavy seepage occurs before the water actually reaches the crop. In some cases farmers use water-tight flumes and pipes for their local distribution, but the proportion of these cases is as yet comparatively small, although on the increase. It has been estimated that seepage losses in the farmers' ditches on many projects is not less than 50% of the losses in the main canal and lateral systems. Allowing for the losses in the farmers' ditches not included in Table 5, the latter might be revised as shown in Table 6, it being understood that the losses in the farmers' ditches are merely the expression of individual opinion, not of actual measurement.

TABLE 6.—PERCENTAGES OF LOSSES.

	Reservoir.	CANAL LOSSES.		Totals.
		Canals and laterals.	Farmers' ditches.	
Umatilla	20	32	15	67
Truckee-Carson	0	41	15	56
Orland	8	23	10	41
Klamath	0	48	15	63
Tieton	0	24	8	32
Sunnyside	0	27	7	34

Seepage losses on the Umatilla Project early assumed serious proportions owing to the sandy character of the soil and the gravelly substrata. With the unlined earthen ditches, as originally constructed, only about one-third of the water diverted reached its proper

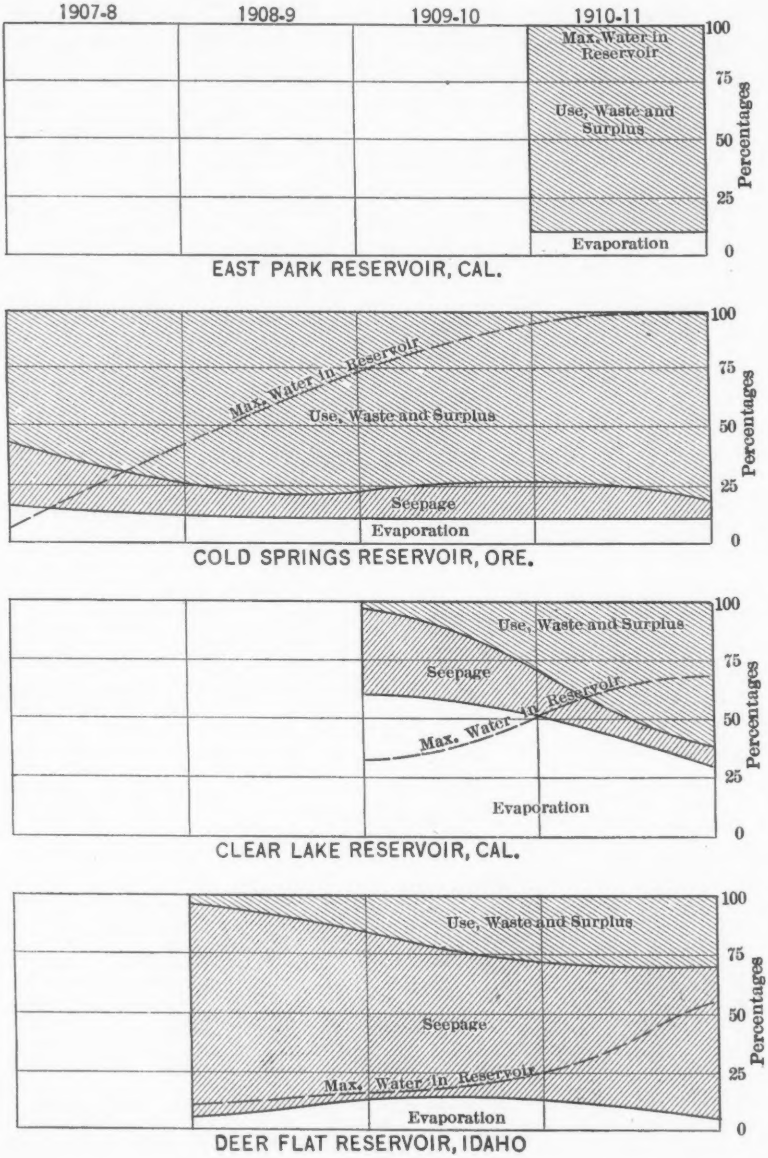
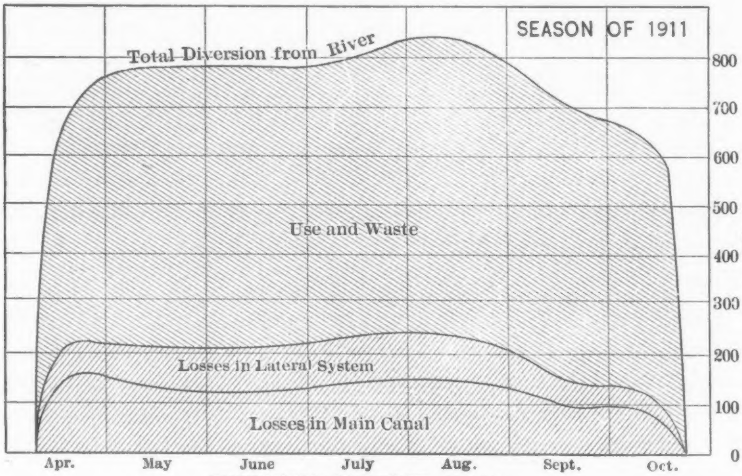
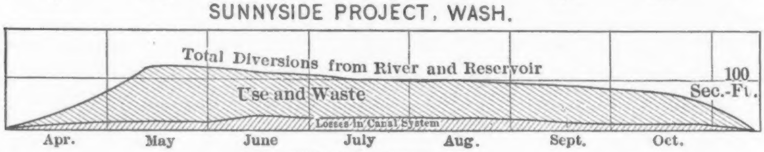

1907-8
1908-9
1909-10
1910-11
Max. Water in Reservoir
Use, Waste and Surplus
Evaporation
100
75
50
25
0
Percentages
EAST PARK RESERVOIR, CAL.
Max. Water in Reservoir
Use, Waste and Surplus
Seepage
Evaporation
COLD SPRINGS RESERVOIR, ORE.
Use, Waste and Surplus
Seepage
Max. Water in Reservoir
Evaporation
CLEAR LAKE RESERVOIR, CAL.
Use, Waste and Surplus
Seepage
Max. Water in Reservoir
Evaporation
DEER FLAT RESERVOIR, IDAHO

FIG. 3.

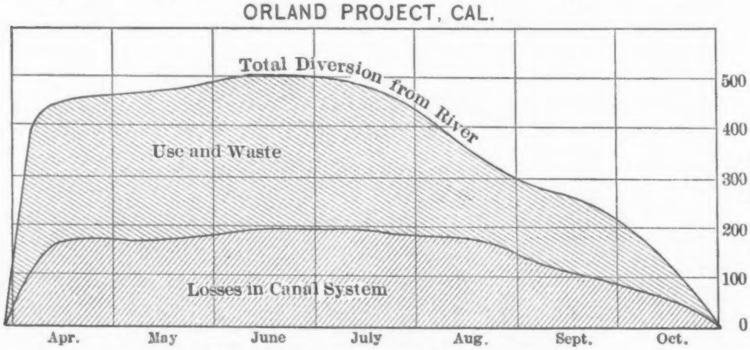
destination. The works were costly, and the quantity of the supply was limited. Unless means could be found to lessen these losses, it was evident that the entire area could not be irrigated, and the building costs would not be wholly repaid.



SUNNYSIDE PROJECT, WASH.



ORLAND PROJECT, CAL.



TRUCKEE-CARSON PROJECT, NEV.

Fig. 4.

About equally severe proportional losses were found on the Truckee-Carson Project, in Nevada, and on the Klamath Project, in Oregon, but in both these projects there is more elasticity, due to their greater

available supplies, and, in the case of the Klamath Project, to the small aggregate quantity used.

Losses on the Tieton and Sunnyside Projects are probably much more satisfactory than in the average well-constructed project in that

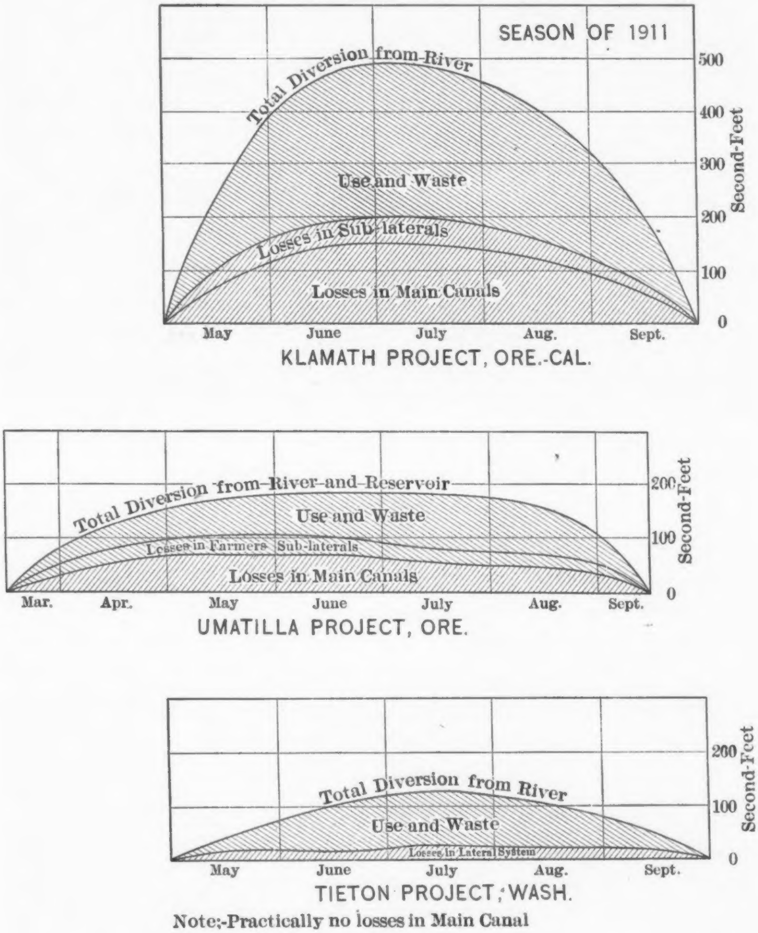


Fig. 5.

vicinity, due, in the first case, to the complete concrete lining of the main canal and the tight character of the substrata of the irrigated lands. In the Sunnyside Project, the relatively small canal losses are due mainly to the fine texture of the soils.

It is probably a fact that in the average project from 40 to 50% of the supply is lost by seepage in the beds of channels before it reaches the actual point of application. As the farmer is paying from $30 to $90 per acre for this water, the loss is very appreciable.

In Southern California valuable orchard lands have been under irrigation for a generation. Crop values have been very high, and, in many cases, the water supply has been so limited that effective measures toward conservation have been enforced. On many projects in that region the distributing channels are lined with concrete, or pipe is used liberally. The high values of lands and the scanty water supply have rendered these measures not only desirable but necessary. Strict economy in use has also been enforced, for the same reason, but, in the newer projects in the Northwest, where crops of lower values obtain, it has not hitherto been seriously regarded as feasible to resort to such expensive treatment. Conditions, however, have changed materially with regard to crop values, and many of the water supplies which appeared to be inexhaustible a few years ago are being rapidly fully appropriated, so that reasons for economy and waste prevention are becoming more and more cogent.

Some interesting experiments carried out under the auspices of the College of Agriculture of the University of California, in 1906, by B. A. Etcheverry, Assoc. M. Am. Soc. C. E., on various kinds of canal lining, including concrete, clay puddle, and oiled surfaces, are worthy of consideration. The object of these experiments was to determine relative costs and efficiencies of different classes of lining in reducing seepage and preventing the growth of vegetation. Without attempting to enter into the details of these experiments,* the general results showed that the concrete lining alone, although the most expensive, gave assured results. The oiling, as would be expected, is very much cheaper than any other treatment, costing only about one-quarter as much per square foot as concrete. During the first year it appears to be of some value in reducing seepage losses, measurements showing that the losses, as compared with those in an untreated earthen canal, are only about 40% of the latter. The oil seemed to be principally valuable in preventing a growth of vegetation. The clay puddle lining gave somewhat better results in preventing seepage

* "Lining of Ditches and Reservoirs to Prevent Seepage Losses," *Bulletin* No. 188, Agricultural Experiment Station, University of California.

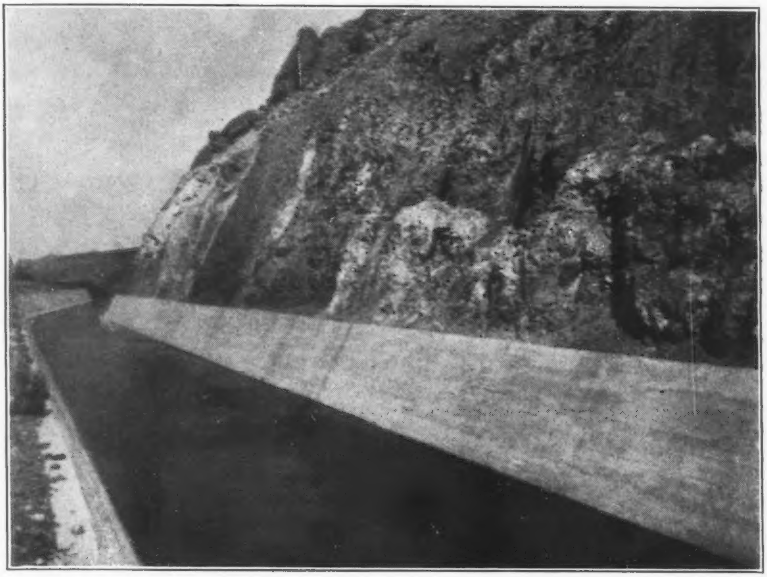


FIG. 1.—MAIN CANAL, TRUCKEE PROJECT, CONCRETE LINED.

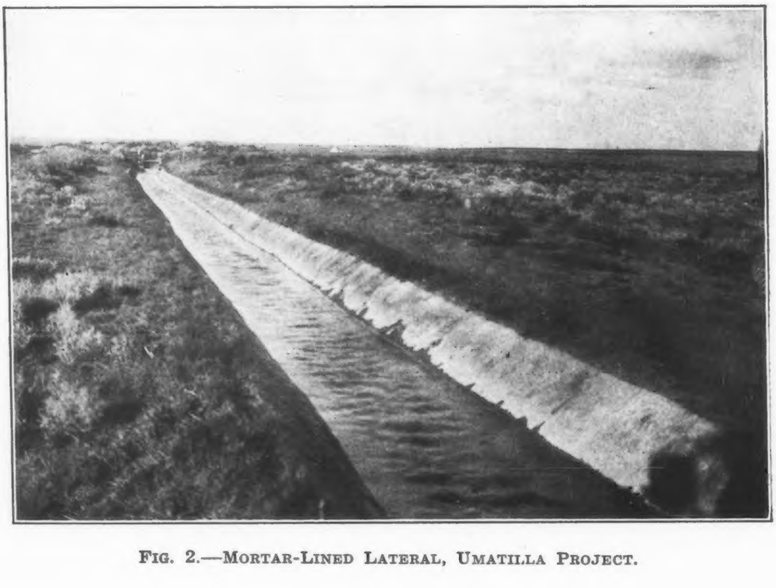


FIG. 2.—MORTAR-LINED LATERAL, UMATILLA PROJECT.

than the oiled surface. The mortar and concrete linings, however, prevented from two-thirds to nine-tenths of the total losses, and, of course, entirely stopped the growth of vegetation.

Apparently, the effect of the oil treatment is only temporary, and a year or two afterward, a re-examination of the canals in which the experiments were made showed that the growth of vegetation in the oil-treated canals was equal to that in those untreated, and in all probability the seepage losses were also as great.

In 1910 and 1911, a lateral on the Umatilla Project was lined with mortar, 1 in. thick, and careful measurements were made to determine the losses. The lateral had been selected for lining on account of the very porous character of its bed and in order to reduce seepage loss. With the lateral closed at the ends by dams, measurements showed that the water surface lowered about 0.1 ft. each day in the lined ditch, and by applying this rate of loss to the canal system as a whole, making due allowance for velocity of flow, it was computed that the aggregate seepage loss in the project, if all the canals were lined, would be about 5% of the supply. With the unlined system the loss is close to 50 per cent. Subsequent measurements have confirmed the above, and, from these and other data, the conclusion has been reached, that seepage losses can be kept down to less than 10% of the amount diverted, if good linings are placed.

During the past two years much canal and ditch lining has been placed on Government projects in the Northwest. These linings are from 1 to 4 in. thick, depending on the size of the canal and the conditions. The heavy linings are of regular sand and gravel concrete having about 1 part of cement to 8 parts of sand and gravel. They are generally placed without forms, the sides of the channel being trued up and a rather dry mix being used. The cost has usually been about $6 per yd. The great bulk of the ditch lining, however, has not been of regular gravel concrete but of mortar, which is usually composed of 1 part of cement to 4 parts of sand. Before placing the mortar the ditches are carefully trued up by running a movable form or templet along their courses, and wetting and tamping the earth around the form. Immediately after the form is removed, the mortar is placed and kept damp until it has set well. It is jointed usually at about 4-ft. intervals in order to take care of temperature shrinkage. This

lining is done with much rapidity by experienced gangs. The materials are mixed in small portable gasoline-driven mixers, and the completed canals are kept full of water. The costs of work of this kind, carried out on a fairly large scale, for lining $1\frac{1}{2}$ in. thick, reinforced at the top by an extra heavy curbing, run from 55 to 60 cents per sq. yd., inclusive of all administrative and engineering charges and of the earthwork. In general, the cost of the earthwork is about one-third of the entire cost.

Take, for example, a small lateral of the Umatilla Project lined in this way during 1911: The length was 12 400 ft.; the ditch dimensions were 4 ft. wide at the bottom, and 4 ft. deep, with side slopes of $1\frac{1}{4}$ to 1; the entire cost of the work averaged $1.05 per ft. Comparing a small ditch thus lined with an unlined one, the former will cost from three to four times more than the unlined ditch, but one of the economic advantages which the lined ditch possesses is the greater velocity of flow possible and the consequently smaller cross-sectional area of the channel. Another important advantage in lined ditches is the avoidance of drop structures. It is surprising what a large proportion of the total cost of ditch building goes into drop structures which are necessary in order to keep velocities below the eroding point in an earthen channel. With lined channels high velocities are not only possible, but desirable, in order to keep the channel clean.

As an illustration of what proportion the cost of structures in a distribution system bears to the entire expense of the latter, figures taken from the Orland Project in California are given. This territory is notably free from topographic irregularities, and the earth is firm and good for building purposes. The proportion of cost of structures, therefore, would be expected to be small.

The lateral system covers 14 000 acres, and includes 54 miles of ditches ranging in capacity from 12 to 75 sec-ft. Very little ditch lining has been placed, but the structures are all of concrete, the cheapest building material. The cost totals are as follows:

Excavation work	$64 376
Structures	57 632
Total	$122 008

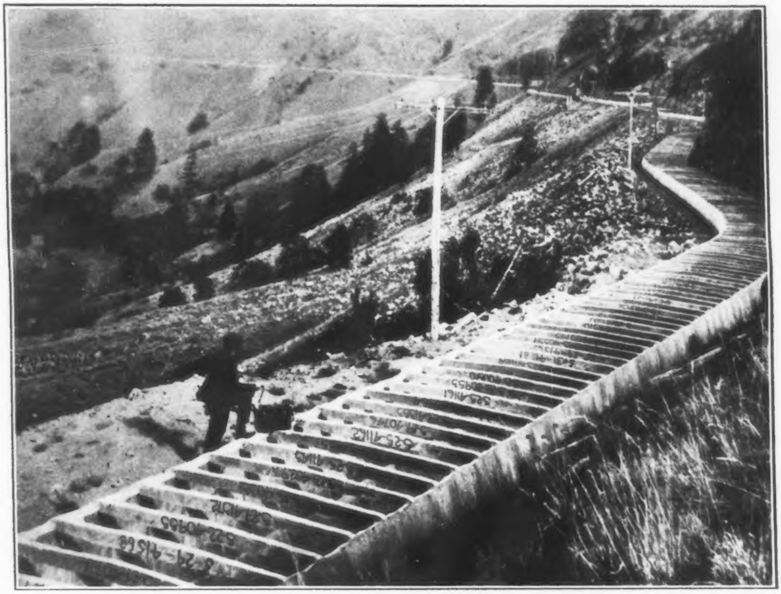


FIG. 1.—TIETON MAIN CANAL, CONCRETE LINED.

FIG. 2.—TYPICAL FARMERS' LATERAL, UMATILLA PROJECT.

The structures included in these cost totals comprise the following types:

Checks and drops	$20 885
Turn-outs	12 901
Bridges	9 972
Railroad crossings	6 924
Special structures	5 805
Spillways	1 143

Checks, drops, and turn-outs total $33 780, or 28% of the entire cost of the lateral system. All this cost could not be obviated by lining the system, but certainly a very large proportion could.

With a smaller cross-sectional area, the saving in drop structures, and the more direct and economical location possible in the lined ditches, the actual difference in cost per acre of land served by lined or unlined canals is comparatively small. It will generally be found to be less than $10 per acre, in many cases less than $5. If one takes into consideration the operating economies, the lined laterals have a distinct advantage by their freedom from breaks, seeped banks, and growth of vegetation in the channels, all of which should admit of a material reduction in operating costs. If these latter savings could be calculated from an investment standpoint and capitalized, any advantage in first cost of the unlined ditches would probably disappear, and a substantial margin be shown on the other side.

While considering canal lining, it would be well to give a little attention to the merits of pipe work in a distributing system. Large quantities of pipe have been used in the distributing systems of the Umatilla, Tieton, and Sunnyside Projects. The great bulk of this pipe is of concrete, both reinforced and plain, in sizes running from 54 in. down to 12 in. in diameter. The sizes below 24 in. have been usually made by the dry process, the reinforcement consisting of outside wire winding under tension. The larger diameters have usually been wet mixed, the pipe being manufactured in yards and hauled and laid like cast-iron pipe. Some of these lines of pipe are of great length and work under heads running up to 110 ft. They have always given satisfaction, from every standpoint. A distribution system consisting wholly of concrete pipe would be undoubtedly the most satisfactory from an operating standpoint, and although the

first cost would be comparatively high, it might in the end prove to be more truly economical than the open-ditch system. With concrete pipe seepage losses are practically negligible. A number of tests of different lines of 4-ft. pipe, under operating conditions, show the following, all this pipe having a shell 3 in. thick:

	Length.	Head.	Average seepage per mile.
1................	4 700 ft.	39 ft.	0.07 sq. ft.
2................	5 400 "	28 "	0.05 " "
3................	3 600 "	19 "	0.04 " "
4................	9 800 "	85 "	0.20 " "

Apparently, the loss per mile in pipe of this size is nearly directly proportional to the head, and averages about 0.02 sec-ft. per mile for each 10-ft. head carried on the pipe. A pipe-distributing system of concrete throughout, under an average pressure head of 50 ft., with delivery to each 40-acre subdivision, would thus only lose about 1%, which is practically negligible.

Taking the average of the six projects quoted, the average cost of the irrigation works would be \$55 per acre with an average combined loss in reservoirs and canals of about 50% of the entire water supply. Of the latter, about 6% is practically incurable reservoir loss; the remaining 44% has been classed as curable, that is, the great bulk of it can be cured or prevented if economical conditions render such action wise.

Should the ditch systems of these projects be wholly lined with concrete or pipe, the losses might be reduced from 44% to 10%, or less, a net saving of 34%, or, say, one-third of the whole supply. It is evident, therefore, that either the systems could be extended to cover about one-third more area, or if such land is not available, the works might be constructed of smaller dimensions and at less cost. In the case of works already built, the latter alternative is inapplicable, and is merely illustrative of what might have been done, but cannot be helped now. The lesson, however, should be applied to new work. In cases where new lands can be taken in under existing works, consideration should be given to the possibilities of extension by lining the present systems.

Suppose, for example, a project of 20 000 acres costing \$55 per acre, or a total of \$1 100 000; if, by lining the ditches, the irrigable

area can be increased to 27 000 acres, there will be first an additional cost for the new laterals with lining, which has been found to be about $18 per acre in a fairly difficult country, or, for the 7 000 acres, an additional construction cost of $126 000 will be necessary. Secondly, there will be the cost of lining the present ditch system, covering 20 000 acres, which, taken at $12 per acre, would mean an added charge of $224 000. The gross cost of the extended project, therefore, would be $1 450 000, or an average cost of $54 per acre. This, apparently, does not result in a material reduction in the acreage cost, but the great advantage lies in rendering available for profitable use the larger areas of land, the conservation of the water supply, and the avoidance of drainage evils referred to later. As a matter of fact, the process of extending a project already constructed with unlined earth canals, by the subsequent lining of ditches, is always much more expensive than if constructed *de novo* with the entire system lined.

In the case of a proposed large extension of the Umatilla Project, it is planned to line the entire canal system from the head-works down to the minor ramifications of the distribution system delivering to each 40-acre subdivision. At no point in the system will the water be exposed to avoidable seepage loss, and when the head-gates at the reservoir are opened, the Government will have the assurance that more than 90% of the supply will actually reach the cultivated fields.

Closely connected with the question of canal losses is the drainage problem. On nearly every irrigation project large and frequently increasing areas will be found subject to the rise of ground-water. The principal contributing influence in most cases is the seepage loss from the lateral systems, although a proportion, of course, is due to over-irrigation of the fields. On the Sunnyside Project, in Washington, some 4 000 or 5 000 acres of the best land was seriously affected a few years ago, large areas having been practically forced out of cultivation. In this case the Government was compelled to build a deep channel at a cost of some $340 000, mainly for the purpose of affording an outlet to the surplus water. On the Minidoka Project, in Idaho, the drainage feature is one of the most serious problems. At Umatilla the seepage water accumulating below the project in the Umatilla River has increased the summer flow some 100 sec-ft., and has rendered necessary the excavation of extensive drainage

ditches through the lower lands. At Klamath some $40 000 has been expended during the past three years on this account, and, on the Truckee-Carson Project, it is planned to expend not less than $400 000 in addition to the large sums already disbursed. There is no question that much relief from this increasing danger will be experienced by eliminating from the ground-water accumulations the bulk of the canal seepage. It is the writer's belief that, as time goes on, it may even be found necessary for legislatures to require canal systems to be lined or otherwise protected from seepage loss, not only in the interests of the investor and water user, but as a reasonable measure of conservation when water supplies are limited. As an engineering and business policy, it is well in the front rank, and should be considered by all who are building new works or operating and extending those already constructed.

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PAPERS AND DISCUSSIONS

SPECIFICATIONS FOR METAL RAILROAD BRIDGES MOVABLE IN A VERTICAL PLANE.*

By B. R. Leffler, M. Am. Soc. C. E.

The excellent paper and specifications for movable bridges by C. C. Schneider, Past-President, Am. Soc. C. E.,† though quite general, pertained mostly to the common swing bridge, or one which rotates about a vertical axis. The writer has felt the need of specifications covering railroad bridges movable in a vertical plane, which necessity was created by the third and fourth track work in progress on the railroad with which he is connected. The common swing bridge is not well adapted to more than two tracks. The writer knows of only two four-track swing bridges in operation.

There seems to be a real necessity among engineers for specifications covering movable bridges of this class. The engineer who has not given long and special study to this class, which is mostly handled by patentees, cannot give adequate consideration to the various designs presented to him under intense competitive conditions. These specifications are intended as an aid to his judgment.

It is not considered wise, at this time, to enter into a discussion of the relative merits of the various patented bridges, the purpose of the specifications being to aid in producing a first-class structure for any style which may be adopted.

Some unsettled technical questions are considered, such as stresses in wire ropes bent over a sheave, the rating and testing of electric

*This paper will not be presented at any meeting, but written communications on the subject are invited for publication with it in *Transactions*.

†*Transactions*, Am. Soc. C. E., Vol. LX, p. 258.

motors, the character of the grooves for the lubrication of trunnions carrying heavy loads, the designing of keys and key-ways, etc.

The writer anticipates that the average mechanical engineer will not agree with the views on stresses in wire ropes. He believes that mechanical engineers use methods which are too loose (under the guise of so-called practical experience) in designing machinery parts. As generally used, wire rope is much over-stressed, principally by being bent over sheaves which are too small. Such practice may do where the rope is readily inspected and easily replaced, but wire rope for supporting counterweights in lift bridges should be designed by formulas which take into account the leading factors affecting the life and strength of the rope. A large factor of safety should then be used.

The rating of electric motors is that adopted by the American Institute of Electrical Engineers, in June, 1907. Some engineers specify a half-hour rating, which usually means a motor of the crane type. Motors for mill work are now being made, and are superior to any other type for strength and ruggedness; these are tested on the one-hour rating, and are suitable for bridge work.

Considerable care should be devoted to the design and workmanship of grooves in large trunnions for lubrication. The grooves should be large and allow of being cleaned. Compression grease cups should be used.

The design and workmanship of keys and key-ways do not usually receive enough attention, as keys come loose and cause damage and delay in the operation of bridges. Erectors sometimes use offset keys, made in the field, for adjusting the relative position of machine parts; but such keys are very objectionable. A key of minimum size, based on the diameter of the shaft and low unit stresses, has been specified.

Cut gear teeth are specified for wheels transmitting considerable power. This is somewhat unusual; but as most railroad bridges are not hand-operated, the resulting smoothness in the running of the machinery is desirable. Cut gears add a very small percentage to the total cost of a structure. The cutting of cast gears sometimes reveals defects which otherwise would remain hidden.

Two formulas are presented for the strength of shafting, axles, etc. This subject is not treated very clearly in works on machine design. The use of the term "equivalent twisting moment" is confus-

ing. The formulas conform to the practice in structural designing of giving a value for tension or compression, and one for shear, respectively.

The writer believes that bridge engineers often specify too high a wind pressure. As usually specified for stationary structures, this includes an allowance for unknown lateral forces which are caused by a train moving over a bridge. Obviously, a smaller wind pressure should be specified for a bridge in motion. A pressure of 10 lb. per sq. ft. means, according to the formula, $P = 0.0032\ V^2$, a velocity of 56 miles per hour. The machinery should be able to hold the structure for a pressure of 15 lb. The wind pressure on a long bascule bridge is a large item.

Designers are sometimes too careless in their methods of designing machinery, relying mostly on rules-of-thumb or so-called experience. All resistance should be separately (and finally in their totality) considered. Coefficients should be adopted for the various sliding and rolling surfaces. The resistance of the moving span and attached parts should be reduced to a single force acting at the rack or in the operating cable. The motor torque for overcoming this resistance, and the machinery resistance, should be shown for all positions of the moving structure. The best method is to plot curves showing the torques, etc.; the time of opening, in 5- or 10-sec. intervals, should be plotted as abscissas, and the motor torque, resistance at rack, etc., as ordinates.

A moving structure is subject to some impact stresses due to its own motion, the magnitude of which cannot be found. The coefficients given simply express the writer's opinion.

No claim of originality is made for all parts of the specifications. The writer is largely indebted to Mr. Schneider's paper; to J. A. L. Waddell and J. L. Harrington, Members, Am. Soc. C. E., for workmanship and material for wire rope and attachments; and to others. To some extent his labors have been those of a compiler.

There is scarcely any first-class technical literature in the United States on the bending of wire rope.*

The writer has endeavored to make the specifications complete, but, of course, this was impossible. Some points are not covered, for instance, a specification should be framed to cover the design of segmental

* Attention is called to the article by Chapman, in the *Engineering Review*, London, October, 1908.

and track girders in rolling bridges, with special reference to taking care of the heavy concentrated load.

A rough test of the power required to open a double-track trunnion bridge of 159-ft. span, weighing, for parts in motion:

Machinery	156 877 lb.	
(Steel) Counterweight....	538 313 "	Counterweight truss-link plate.
Span	797 009 "	Operating struts.
Concrete	2 570 000 "	
	3 062 199 lb. = 1 531 tons.	

TEST OF POWER:

Controller notches.	Volts.	Amperes.	
1	212	150	Time required to open = 2 min. 20 sec.
2	210	200	
3	210	200	
4	210	250	
5	210	200	
6	210	190	
7	201	350	
8	201	225	
9	201	375	
10	201	300	
1	219	60	Time required to close = 2 min.
2	219	60	
3	219	60	
4	219	75	
5	219	75	
6	219	75	
7	205	250	
8	205	225	
9	205	200	
10	205	200	
Average of readings of another test.	205	240	Time required to open = 2 min. 15 sec.
	216	93	Time required to close = 2 min.

SPECIFICATIONS FOR BRIDGES MOVABLE IN A VERTICAL PLANE.

1.—These specifications are intended to cover bascule bridges, which are such as rotate about a horizontal axis; and vertical lifts, which are those in which successive positions are parallel. **Scope.**

2.—The specifications of The New York Central Lines for Steel Railroad Bridges, for 1910, shall apply to movable bridges, except as noted herein.

Manner of Bidding.

3.—Drums, cylinders, eccentrics, trunnions and their cast supports, shafting, pistons, gear wheels, racks, boxings, bearings, couplings, disks, cast sheaves and wheels, worm gearing, valves, pins about the axis of which the connecting members rotate, whistles, ram screws, end bridge locks, rail locks, indicators, cranks, axles, hooks, wrenches, and similar parts of machinery which require machine-shop work, shall be classified as machinery and be paid for at a common price per pound. Electric motors are not classified as machinery. **Parts Classified as Machinery.**

4.—The large sheaves of vertical lift bridges, the webs and diaphragms of which are built up with plates, angles, and rivets, shall be paid for at a separate price per pound of finished weight including casings and fastenings to trunnions. **Sheaves.**

5.—Air compressor tanks and steam boilers shall be paid for at a separate price. **Air Compressor Boilers.**

6.—Wire ropes and cables shall be paid for at a separate price per pound. **Wire Ropes and Cables.**

7.—The sockets, pins, equalizing levers, and cable attachments to the trusses and counterweights shall be paid for at a separate price per pound. **Sockets, Pins, Levers, etc.**

8.—Structural steel supporting the machinery proper, counterweight frames, counterweight trusses, towers, and links shall be classified as structural steel and be paid for at the same price per pound as for the span itself. **Structural Steel Parts.**

9.—Structural steel which can be fabricated by the common shop methods as punching, reaming, drilling, shearing, planing, etc., as is usually done for stationary structures, shall be classified as structural steel and be paid for at the same price per pound as for the span itself.

10.—Segmental girders in rolling bascule bridges and the horizontal girders on which they roll shall be paid for at a separate price per pound. This does not include any bracing, floor system, or other structural members which may be attached. **Segmental Girders.**

11.—Electric equipment, such as wiring, switch-boards, controllers, lights, blow-outs, cut-offs, solenoids, switches, motors, etc., shall be paid for on a lump-sum basis. **Electric Equipment.**

12.—Cast-iron parts used in counterweights shall be paid for at a separate price per pound.

13.—Concrete in counterweights shall be paid for at a price per cubic yard in place.

Extra Parts, etc.

14.—It is to be understood that if any extra parts are needed, or any question arises, all difficulties shall be settled on the pound price basis as quoted and accepted for the parts in question.

General Details of Designing.

Self-Centering Devices.

15.—Self-centering and seating devices shall be used on the free ends of the moving span. Holding and forcing-down devices shall be used for the free ends of each truss.

Rail Locks.

16.—Designs for bridging the gap between the shore rails and moving rails shall be furnished by the Railroad Company. Loose rails will not be allowed.

Air Buffers.

17.—Air buffers shall be furnished at the free ends of the moving span.

Counterweights.

18.—The counterweights shall be easily adjustable. Usually, this shall be done by adding or taking away cast-iron parts, or small concrete blocks.

Stairways.

19.—Metal stairways, with $1\frac{1}{2}$-in. hand-rail, shall be provided, for access to the machinery, trunnions, and counterweights.

Girders in Rolling Bridges.

20.—The reinforcements of webs in the segmental girders and track girders of rolling bridges shall be symmetrical about the center planes of the webs. The center planes of the segmental webs shall coincide with the corresponding center planes of the webs of the track girders.

Coefficients of Friction for Moving Span and Attached Parts.

21.—In calculating the resistances to be overcome by the machinery, the resisting forces shall be reduced to a single force acting between the pinion and operating rack, or in the operating cable. In determining this force, the following coefficients shall be used in starting the span, and, except for the stiffness in cables, shall be reduced one-half after motion is begun:

For friction on trunnions	$\frac{1}{8}$
For rolling friction of rolling bridges	$\frac{1}{12}$
For stiffness in cables	$\frac{1}{200}$

Losses in Operating Machinery.

22.—In figuring the machinery losses between the operating rack or operating cable and the motor, the following coefficients shall be used: for the efficiency of any pair of gears, 0.94; for journal friction, 0.07. The losses of any worm gear shall be taken at 30% for an angle of thread 20° or more.

Time to Open.

23.—The time to open the bridge after the ends are released shall be as specified on the proposed drawing.

24.—The force necessary to overcome the inertia and produce acceleration and retardation for the time of opening shall be considered. The machinery shall be capable of stopping the bridge in 6 sec.; for this purpose, the coefficient of friction in the friction brake shall be taken at not less than 25 per cent. Inertia.

25.—In calculating the dead-load stresses in the moving structural parts, for the various positions of the open bridge, such stresses shall be increased 25% as allowance for impact. For stationary structural parts (as towers, and supporting girders in rolling bridges), to which moving parts are attached, or on which such parts roll, 15% of the static load shall be added as impact. Impact in Structural Parts.

26.—In structural steel parts, where a percentage of the dead load or static load is added for impact, the unit stresses for stationary structures shall be used; the impact percentages are an allowance similar to that provided by an impact formula for stationary railroad bridges.

27.—The allowance for impact in trunnions, cables, cable attachments, and machinery parts is taken care of by lowered unit stresses. Impact for Machinery Parts, etc.

28.—The least wind pressure to be assumed in proportioning the machinery or moving parts shall be 15 lb. per sq. ft. on the exposed surfaces of the moving parts as projected on any vertical plane. The machinery shall be strong enough to hold the moving parts in any position for this pressure, and be capable of opening the bridge in the specified time at 10 lb. per sq. ft. wind pressure. Wind Pressure.

28a.—On the ordinary open-floor bridge with ties, the exposed surface to wind shall be taken equal to 80% of a full quadrilateral the width of which is the distance from center to center of trusses and the length of which is that of the moving span.

29.—The Contractor shall make complete detailed drawings of the machinery, so that any other shop can take them and duplicate the machinery. No reference to patterns or individual shop practices will be considered in lieu of the complete drawings. These drawings shall show a general outline of the assembled machinery. The drawings shall be made on tracing cloth, each sheet 24 by 36 in. in outside dimensions. These drawings shall become the property of the Railroad Company on the completion of the job. Detailed Drawings.

30.—The Contractor shall furnish an outline drawing of the machinery, on which are shown the forces acting on the gear teeth, the twisting moment and bending moment on shafts, and other necessary information for checking the strength of the machine parts. A tabulation of the formulas and methods of calculation shall be shown complete enough to allow them to be checked. Outline Drawing of Machinery.

31.—The Contractor shall show by a drawing of curves the torque to be exerted by the motor or prime mover, as follows: Torque Curves.

1. A torque curve for acceleration and retardation;
2. A torque curve for the frictional resistances;
3. A torque curve for any unbalanced condition of the structure;
4. A torque curve for the wind load;
5. A torque curve showing the greatest combination of resistances acting at any one time.

In figuring the friction at starting (this being twice the running friction), no acceleration of the moving mass shall be considered. This friction shall be considered as reduced to the running friction in the first second after the power is applied.

Center of Gravity.

32.—The Contractor shall check the location of the center of gravity of the moving span, including all parts attached thereto, and also the location of the center of gravity of the counterweight, including counterweight girders and trusses, by computations based on accurate weights calculated from shop plans. He shall submit duplicate sketches and copies of these computations accompanied by weight bills to the Railroad Company for approval.

Hand Operation.

33.—All bridges shall be equipped with hand-operating mechanism. The number of men and the time required to operate shall be estimated on the assumption that the force one man can exert on a lever is 40 lb. with a speed of 160 ft. per min. developing about $\frac{1}{5}$ h.p. For calculating the strength of the machinery, the power of one man shall be assumed as 125 lb., but 150 lb. shall be the minimum used and applied to the extreme end of a lever.

Operating Machinery.

34.—The parts shall be simple in design, and easily erected, inspected, adjusted, and taken apart. The fastenings shall hold the parts in place securely after they have been set.

Kind of Material.

35.—Rolled or forged steel shall be used for bolts, nuts, keys, cotters, pins, axles, screws, worms, piston rods, trunnions, and crane hooks, if any.

36.—Trunnions, pins, and shafting more than $3\frac{1}{2}$ in. in diameter shall be of forged structural steel. Shafting $3\frac{1}{2}$ in. or less in diameter may be of cold-rolled steel.

37.—Forged or cast steel shall be used for levers, cranks, and connecting rods.

38.—Cast steel, or forged steel, shall be used for couplings, end shoes, racks, toothed wheels, brake wheels, drums, sheaves, and hangers where the supported weight will cause tensile stresses. Large sheaves may be built of structural steel.

39.—Pinions shall be made of forged steel, and cut from the solid metal.

39a.—Pinions shall have not less than fifteen teeth.

40.—Sockets used for holding the ends of wire ropes shall be forged without welds, from the solid steel.

Cast Iron. 41.—Cast iron may be used in boxes for shafts 2 in. or less in diameter, and which obviously carry light loads. Other boxes shall be of cast steel.

42.—Cast iron may be used in eccentrics, cylinders, pistons, fly wheels, and parts of motors which are usually made of cast iron. Cast iron shall not be used for any trunnion or axle support.

Metal for Bushings. 43.—Phosphor-bronze, brass, and Babbitt metal shall be used for the bushing or lining of journal bearings and other rotating or sliding surfaces, to prevent seizing.

44.—Phosphor-bronze, only, shall be used for bushing for the trunnions of bascule and lift bridges, or in any large bearing carrying heavy loads.

45.—The bushings for large bearings, such as for trunnions and similar parts, shall be held from rotating in their casings. The force tending to cause rotation shall be taken as one-eighth of the load on the trunnion or bearing and as acting tangent to the surface between the back of the bushing and casing. It shall be practicable to take out the bushing when the trunnion is slightly lifted.

Castings. 46.—Castings which are to be attached to rough unfinished surfaces shall be provided with chipping strips. The outer unfinished edges of ribs, bases, etc., shall be rounded off, and inside corners filleted.

Bolts and Nuts. 47.—Bolts and nuts, up to $1\frac{1}{2}$ in. in diameter, shall have U. S. Standard V-threads. Nuts and exposed bolt heads shall be hexagonal in shape, and each nut shall be provided with a washer. If the nut will come on an inclined surface, a special seat, the top surface of which is at right angles to the bolt, shall be cast or built up to receive the nut. Bolt heads which are countersunk in castings shall be square.

48.—Nuts which are subject to vibration and frequent changes of load shall have locking arrangements to prevent the gradual unscrewing of the same. If double nuts are used for that purpose, each nut shall be of the standard thickness. Nuts shall be secured by split pins put through the bolt.

Screws. 49.—Screws which transmit motion shall have square threads.

Tap-Bolts, Set-Screws, etc. 50.—Tap-bolts and stud-bolts shall not be used, except by special permission.

51.—Set-screws shall not be used for transmitting torsion to shafts or axles. They shall be used for holding keys, or other light parts, in place.

Collars. 52.—Collars shall be used wherever necessary to hold the shaft from moving horizontally. Each collar shall have at least two set-screws at an angle of 120 degrees.

Shaft Couplings. 53.—Shaft couplings, unless of the flexible kind, shall be of the flange type, or split muff with bolt heads and nuts countersunk.

54.—Couplings shall be keyed to shaft.

Keys. 55.—Gib-head or hooked keys shall be used for keying machinery parts to shafts, except where otherwise shown. The keys shall have the proportions shown in Fig. 1, in which d is the diameter of the shaft.

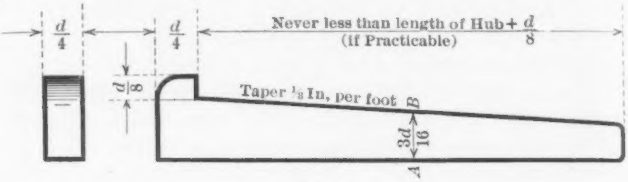

FIG. 1.

$A B$ is a mid-section of the tapered length. The sides shall be parallel.

56.—If the foregoing shape of key gives unit stresses in shear or bearing exceeding those in the table of allowable unit stresses, its section must be increased.

57.—The key shall be sunk in grooves in both hub and shaft. The finish of the grooves and key shall be such as to give a full bearing on all four sides, except as taper of key will not allow.

58.—If practicable, the groove in the shaft shall be made long enough to allow the key to be inserted without moving the wheel sidewise. After the key is firmly seated, the groove shall extend beyond the point of the key a distance not less than $\frac{3\,d}{8}$ to allow for future tightening; the clear distance between hub and hook of key shall not be less than $\frac{d}{8}$.

59.—The depth of the groove in the shaft shall be $\frac{3\,d}{40}$, measured at the side of the groove.

60.—In the case of large shafts carrying heavy parts, two or more keys of special design shall be used. In such cases, the matter shall be taken up with the Engineer, for special study.

61.—The foregoing requirements for keys and key-ways are for major machinery parts, the use of which is intended to develop the full torsional strength of the shaft. For minor parts, the keys and key-ways shall be proportioned for that size of shaft in which torsional strength would be developed by the minor parts.

62.—Keys shall be held in place by set-screws. **Set-Screws for Keys.**

63.—If practicable, the length of the hub shall be not less than $2d$. Its thickness shall be not less than $\frac{d}{3}$. The hub shall have a light driving fit. **Hub.**

64.—The groove in the hub shall be made on the center line of an arm.

65.—Hubs shall be bored truly at the center of the wheel.

66.—For trunnions and similar parts, which are designed chiefly for bending and bearing, the keys, key-ways, and bolts shall be designed to hold the trunnion from rotating. The force tending to cause rotation shall be taken at one-fourth the load on the trunnion, and shall be taken as acting at the circumference of the trunnion. **Keys in Trunnions.**

67.—Journals shall be proportioned to resist, not only the various stresses to which they are subjected, without exceeding the permissible fiber and bearing stresses, but also to prevent a tendency to heat and seize. **Journals.**

68.—Steel bearings carrying steel shafts or journals shall be lined with bronze or brass. If shafts are 3 in. or less in diameter and of a slow motion, Babbitt metal may be used. Bearings of steel on steel for moving surfaces will not be allowed. **Bushings.**

68a.—Divided journal and trunnion bearings shall be used, and the cap shall be fastened to the base with turned bolts recessed into the base. The nuts and heads shall bear on finished bosses cast on the bearing.

69.—In cast-iron boxes carrying light shafts, no lining is needed. **Boxes.**

70.—The bearings of shafts shall be placed as near to the points of loading as possible.

71.—The foot-steps of vertical shafts shall be of axle or tool steel, and shall run on bronze disks.

72.—Provision shall be made for the effective lubrication of journals, or any other sliding surfaces. Closed oil or compression grease cups shall be used. Grooves shall be cut in the surface of the trunnion to provide for the proper distribution of grease or oil. **Lubrication.**

73.—The grooves in large trunnions shall approximate to a U shape; the size shall be such that a wire $\frac{5}{16}$ in. in diameter may lie wholly within the groove. The edge of the U shall be rounded to a radius of $\frac{3}{16}$ in. **Grease Grooves.**

74.—The grooves shall be straight, running parallel to the axis of the trunnion. They shall be not less than three in number, and located so that all parts of the bearing surface of the bushing will be swept by the contained lubricant in an opening, and in a closing of the bridge. The grooves must allow of being cleaned with a wire.

75.—In any trunnion bearing, or similar heavy bearings, strong compression grease cups shall be used for the grooves. **Grease Cups.**

76.—Oil and grease ducts shall be located so that the lubricant will flow by gravity toward the bearing surface.

Dust Covers.

77.—Dust covers shall be provided for principal bearings, in particular for trunnions.

Shaft Supports and Couplings.

78.—Line shafts, extending from the center of the bridge to the end, shall not be continuous, but shall be connected with claw couplings. Each length of shafting shall rest in not more than two bearings, with the couplings close to the bearings.

79.—If shaft supports are connected to the floor-beams, in bridges having long panels, intermediate supports shall be used; these shall be adjustable, and are intended merely to prevent the shaft from sagging.

Equalizing Gears.

80.—Equalizing gears or devices shall be used to insure equal action at the pinions and operating racks.

Unsupported Length of Shafts.

81.—The unsupported length of shafts shall not exceed $L = 80\sqrt[3]{d^2}$ for shafts supporting their own weight only; $L = 50\sqrt[3]{d^2}$ for shafts carrying pulleys, gearing, etc., where L = length of shaft between center of bearings, in inches; and d = diameter of shaft, in inches.

82.—Line shafts connecting machinery at the center to that at the ends shall run at fairly high speed. The speed reduction shall be made in the machinery near the end.

83.—In designing circular shafting, trunnions, and axles, the greatest unit fiber stress in tension or compression due to bending shall be calculated by the following formula:

$$f = \frac{32}{\pi\, d^3}\left(\frac{3}{8} M + \frac{5}{8}\sqrt{M^2 + T^2}\right).$$

Formulas for Shafts.

84.—The maximum unit shear shall be calculated by the following formula:

$$S = \frac{16}{\pi\, d^3}\sqrt{M^2 + T^2}.$$

85.—In these formulas, f = unit fiber stress in tension or compression; S = unit shear; d = diameter of shaft; M = the simple bending moment, and T = the simple twisting moment.

Effect of Key-Ways in Shafts.

86.—If a shaft, trunnion, or axle has one key-way cut, f and S shall be increased by one-sixth; if two key-ways are cut, increase by one-fourth. If the shaft, etc., is enlarged through the hub, this does not apply.

Distance Between Shaft Supports.

87.—In calculating the bending moment on shafts, trunnions, and journals, the distance from center to center of bearings shall be taken.

Style of Gear Teeth.

88.—Gear teeth shall be of the involute type, with an angle of obliquity of 20 degrees. The roots below the clearance line shall be filleted.

89.—The width of the teeth may be as great as four times the pitch, but not more, except for wheels running at a very high velocity, as in motors where abrasion is to be considered.

Strength of Beveled Gear Teeth. 90.—In estimating the strength of teeth in bevel wheels, the pitch at the middle section shall be taken.

Pitch Circle. 91.—For the purpose of setting gear teeth accurately in the field erection, the pitch circle shall be scribed on the ends of the teeth.

Worm Gearing. 92.—Worm gearing, for transmitting power, shall have an angle of thread not less than 20 degrees. The worm shall run in oil. A bronze or brass collar shall be used at the end of the worm and at the end of the wheel axle, to take care of the end thrust. The wheel shall be of bronze. If a nut engages the worm, the nut shall be of bronze.

92a.—Worm wheels shall have not less than twenty-eight teeth.

93.—Worms which are to be used for actuating signals, indicators, or other minor parts may have an angle of thread less than 20 degrees.

93a.—Safety guards shall be provided around gears and other moving parts where it is necessary for workmen to be while the machinery is in motion.

Counterbalancing, Operating Ropes, and Attachments.

Wire Ropes and Cables. 94.—Wire rope shall be made by some manufacturer approved by the Engineer.

95.—The counterbalance ropes shall be of plow-steel wire, and shall consist of six strands, of nineteen wires each, laid around a hemp center.

96.—Ropes shall be laid up in the best manner, and shall be thoroughly soaked in an approved lubricant during the process of manufacture.

97.—The counterbalance ropes shall be made from wire which has been tested in the presence of an inspector, designated by the Engineer, and which, for sizes from 0.76 to 0.150 in. in diameter (the limiting values used in counterbalance ropes), exhibits the following physical properties:

a.—The tensile strength shall be not less than 225 000 lb. per sq. in. for wire from 0.150 to 0.126 in., nor less than 230 000 lb. for wire from 0.125 to 0.101 in. in diameter; nor less than 235 000 lb. for wire from 0.100 to 0.076 in. in diameter.

b.—The total ultimate elongation, measured on a piece 12 in. long, shall be not less than 2.4 per cent.

c.—The number of times a piece 6 in. long can be twisted around its longitudinal axis without rupture shall be not less than 1.4 divided by the diameter, in inches.

d.—The number of times the wire can be bent 90° alternately to the right and to the left, over a radius equal to twice its diameter, without fracture shall be not less than six. This test shall be made in a mechanical bender constructed so that the wire actually conforms to the radius of the jaws and is subjected to as little tensile stress as possible.

Ultimate Strength of Cables.

98.—The rope shall be made in one piece, if possible. Its breaking strength, as determined by the test described in Paragraph 101, shall be not less than

4 900	lb.	if	$\frac{1}{4}$	in.	in diameter.
11 800	"	"	$\frac{3}{8}$	"	" "
20 600	"	"	$\frac{1}{2}$	"	" "
32 400	"	"	$\frac{5}{8}$	"	" "
45 000	"	"	$\frac{3}{4}$	"	" "
70 200	"	"	$\frac{7}{8}$	"	" "
79 200	"	"	1	"	" "
100 800	"	"	$1\frac{1}{8}$	"	" "
120 600	"	"	$1\frac{1}{4}$	"	" "
148 000	"	"	$1\frac{3}{8}$	"	" "
173 000	"	"	$1\frac{1}{2}$	"	" "
200 000	"	"	$1\frac{5}{8}$	"	" "
230 000	"	"	$1\frac{3}{4}$	"	" "
264 000	"	"	$1\frac{7}{8}$	"	" "
297 000	"	"	2	"	" "
325 000	"	"	$2\frac{1}{8}$	"	" "
374 000	"	"	$2\frac{1}{4}$	"	" "
465 000	"	"	$2\frac{1}{2}$	"	" "

99.—In case the breaking strength of the rope falls below the values cited above, the entire length from which the test pieces were taken shall be replaced by the manufacturer with a new length, the strength and physical qualities of which come up to the specifications.

100.—Sockets used in connection with this rope shall be forged, without welds, from solid steel. In every case the dimensions shall be such that no part under tension shall be loaded higher than 65 000 lb. per sq. in. when the rope is stressed to its ultimate strength, as named above. The sockets must be attached to the rope by a method which is absolutely reliable and will not permit the rope to slip in its attachment to the socket.

101.—In order to show the strength of the rope and fastenings, a number of test pieces, not more than 10% of the total number of finished lengths which will be ultimately made, nor less than two from each original long length, and not more than 12 ft. long, shall be cut, and shall have sockets, selected at random from those which are to be used in filling the order, attached to each end. These test pieces are to be stressed to destruction in a suitable testing machine. Under this stress the rope must develop the ultimate strength given in Paragraph 98.

102.—The sockets must be fastened to the rope so that there is no slipping of the rope in the basket. If slipping should occur, then the method must be changed until one is found whereby slipping can

be entirely avoided. The sockets themselves shall be stronger than the rope with which they are used; if one should break during the test, then two others shall be selected and attached to another piece of rope and the test repeated; and this process shall be continued until the inspector is satisfied of their reliability, in which case the lot shall be accepted. If, however, 10% or more of all the sockets tested break at a load less than the minimum ultimate strength of the rope given in Paragraph 98, then the entire lot shall be rejected and new ones shall be made of stronger material.

103.—The length of each rope, from inside of bearing to inside of bearing of sockets, shall be determined, and a metal tag having the said length stamped thereon shall be securely attached to the rope. **Length of Rope.**

104.—The purchaser reserves the right to test each wire rope connection, after its attachment is made, up to one-half of the ultimate strength of the rope, and, if it shows the least sign of weakness, it shall be rejected and replaced.

105.—The manufacturer shall provide proper facilities for making the tests, and shall make at his own expense all the tests required. Tests shall be made in the presence of an inspector who represents and is paid by the Engineer. **Facilities for Testing Rope.**

106.—Ropes shall be shipped in coils the minimum diameter of which is at least thirty times that of the ropes, and they shall be uncoiled for use by revolving the coil, not by pulling the rope away from the stationary coil. **Shipment of Rope in Coils.**

107.—The equalizing levers connecting the ropes to the counterweights and their pins more than 3½ in. in diameter shall be of forged steel; pins 3½ in. in diameter or less shall be of rolled machinery steel. The levers shall be neatly finished, and shall conform to the dimensions shown on the drawings. **Equalizing Levers.**

Workmanship.

108.—For the parts of the operating machinery of movable bridges which are usually exposed to the weather, the finish shall be confined to the bearing, rotating, and sliding surfaces, and wherever it is required to produce accurate fits and precise dimensions.

109.—Castings shall be cleaned, and seams and other blemishes removed.

110.—Drainage holes, not less than ¾ in. in diameter, shall be drilled in places where water is likely to collect.

111.—Unfinished bolts may have a play of $\frac{1}{16}$ in. in the bolt holes. Turned bolts must have the diameter of the shank at least $\frac{1}{16}$ in. larger than the diameter of the threaded portion, and must have a driving fit in the bolt hole. **Play in Unfinished Bolts.**

112.—The backs of racks and contact surfaces shall be planed. **Racks and Contact Surfaces.**

Tread Plates. 113.—The top and bottom of the tread plates and contact surfaces in rolling bridges shall be planed to fit. A full bearing must be made.

114.—The periphery and the ends of teeth which mesh with a shrouded pinion shall be planed, and the pitch line scribed thereon.

115a.—The joints between the caps and bases of journal and trunnion bearings shall be planed. The ends of the bases and surfaces in contact with the supports shall be planed. Bolt holes for holding the cap to the base and for holding the base to its support shall be drilled.

Finishing of Trunnions, etc. 115.—Journals and trunnions shall be turned with a fillet at each end and at points where the section changes. Trunnions and journals 8 in. and more in diameter shall have a hole, $1\frac{1}{2}$ in. in diameter, bored through on the longitudinal axis. Journals, trunnions, and bushings must be polished after being turned. The use of a cutter which trembles or chatters will not be allowed.

Grooves. 116.—The grooves in the surfaces of trunnions or similar large bearings shall be machine cut. Chipping and filing will be allowed only for removing small inequalities. The grooves shall be smooth, especially the rounded corners.

Hubs. 117.—Hubs of wheels, pulleys, couplings, etc., shall be bored to fit close on the shaft axle. If the hub performs the function of a collar, the end next to the bearing shall be faced. Holes in hubs of toothed gear wheels shall be concentric with the pitch circle.

Cut Gears, etc. 118.—The periphery of gear wheels shall be turned. Gear wheels which are part of the train which actuates the moving span, or the bridge locks, or the rail locks, shall be cut. Machine-moulded teeth may be used for actuating signals or small parts.

Beveled Gears. 119.—Beveled gears shall be cut. The cutting shall be done by a planer having a rectilinear motion to and from the apex of the cone. Rotating milling cutters shall not be used.

Grooves in Sheaves. 120.—The grooves in the circumference of sheaves carrying wire ropes shall be turned to a radius which will fit the rope. This is to be done after the sheave is completely assembled and permanently riveted up.

121.—At the juncture of the shrouding and teeth in pinions, cleaning, chipping, or other means shall be used to insure the meshing of the pinion teeth and rack teeth.

122.—Threads on worms, and the teeth of worm wheels shall be cut and shall fit accurately. Point contact shall be avoided.

123.—Any two surfaces which slide, roll, or bear on each other shall be planed.

Assembling of Machinery. 124.—Machinery parts shall be assembled on the supporting members in the shop, and shall be aligned and fitted, with holes in the supports drilled, and with the members in correct relative position. The members shall be match-marked both to the supports and to each

other, and re-erected in the same relative position; or, if not assembled in the shop, connecting holes in the supports shall be drilled in the field.

125.—The holes in the girders and columns for the bolts connecting the main sheave bearings to their supporting girders shall be drilled from the solid through cast-iron or steel templets on which the bearings were set and accurately lined when the holes in the bearing were bored. The bolt holes and the bolts shall be turned to the same diameter and the bolts driven to place without injury to them, the bearings, or the girders or columns. **Holes for Sheaves for Vertical Lift Bridges.**

126.—If trunnions rotate in fixed pedestal bearings, such as the sheave trunnions in vertical lift bridges or similar bearings, the pedestals shall be firmly mounted in the shop, the trunnions placed therein and covers bolted, the whole, when assembled, shall simulate the assemblage in the field as nearly as practicable. The maximum torque in inch-pounds required to rotate the trunnion shall be $\frac{W\ r}{10}$, where W equals the weight of the trunnion, in pounds, and r equals the radius of the trunnion, in inches. If large structural parts rotate about the axis of the trunnion, the trunnion shall be inserted in its bushing in the structural part and rotated. If the shop position of the structural part is flat, which is the usual case, the axis of the trunnion will be vertical, and there will be no load on the bearing; in this case the maximum torque required to rotate the trunnion shall be $\frac{W\ r}{50}$. At least four complete rotations of the trunnion must be made. If any grinding or hard turning is found, it must be remedied. These trunnion tests shall be made in the presence of the Railway Company's inspector and with such apparatus as will readily determine the torque. **Shop Test on Trunnions.**

127.—Faces of flange and split muff couplings shall be planed to fit. The couplings shall be keyed to the shaft. **Facing of Couplings.**

128.—A special effort to secure good workmanship on keys and key-ways shall be made.

129.—Machined surfaces shall have a coating of white lead applied to them. **Coating of Surfaces.**

130.—Machinery which is of the regular standard manufactured type, such as steam, gasoline, electric motors, pumps, air compressors, etc., shall be guaranteed by the manufacturer as to efficiency, and shall be subject to the approval of the Engineer. Motors shall be tested to prove that they fulfill the specified requirements and develop the desired speed, power, and torque.

131.—The rating of a motor shall be the horse-power determined by the brake test. **Brake Test of Motors.**

A. I. E. E. Rules.

132.—The electric equipment shall conform to the Standardization Rules of the American Institute of Electrical Engineers, as approved June 21st, 1907. (See "Standard Hand Book for Electrical Engineers," 3d Edition, Sect. 19.)

133.—The unit stresses per square inch, to be used for parts in which main stresses are not increased by impact, shall be as follows:

STRESSES IN ONE DIRECTION, IN POUNDS PER SQUARE INCH.

Material.	Tension.	Compression.	Fixed Bearing.	Shear.
Machinery steel.........	9 400	$9\,400 - 40\frac{l}{r}$	11 000	6 200
Structural steel..........	8 500	$8\,500 - 36\frac{l}{r}$	......	5 600
Steel castings...........	7 000	$8\,000 - 35\frac{l}{r}$	......	5 000
Phosphor-bronze.........	6 600	....	......	4 600
Cast iron	3 000	8 000	......	3 000
Shear on keys...	.....	4 900 lb.		
Bearing on keys.	.....	8 800 "		

134.—The maximum unit tension in plow-steel cables shall be one-sixth of the ultimate. The maximum unit tension is equal to the direct unit stress plus the extreme fiber unit stress in the individual wire due to bending over the sheave.

Reversal of Stresses.

135.—For stresses which are reversed at the rate of five or more times per minute, use one-half of the above unit stresses.

136.—If wire rope is bent over a sheave, the bending stress and permissible load on the rope shall be calculated as follows:

Let P = the total pull or permissible load, on the rope, in pounds;
K = extreme unit fiber stress in the greatest individual wire;
E = modulus of elasticity = 28 500 000;
a = cross-sectional area of rope, in square inches;
d = diameter of thickest wire, in inches;
D = diameter of sheave to center of rope, in inches;
S = greatest unit tension allowable;
α = angle of helical wire with axis of strand;
β = angle of helical strand with axis of rope;
c = diameter of rope.

$$\text{Then } K = \frac{Ed \cos.^2 \alpha \cos.^2 \beta}{D} \quad \ldots\ldots\ldots\ldots\ldots (1)$$

$$P = a\left(S - \frac{Ed \cos.^2 \alpha \cos.^2 \beta}{D}\right) \quad \ldots\ldots\ldots\ldots\ldots (2)$$

For rope having six strands of nineteen equal wires each,

$$P = a\left(S - \frac{1\,800\,000\,c}{D}\right) \ldots\ldots\ldots\ldots\ldots\ldots \quad (3)$$

because $cos.^2\ \alpha\ cos.^2\ \beta = 0.95,\ d = \frac{c}{15}$.

137.—For haulage rope, six strands of seven wires each, take $d = \frac{c}{9}$.

138.—If a rope is in contact with a sheave over a small arc, the actual radius of curvature may be greater than that of the sheave. (Fig. 2.)

Let R = the actual radius of curvature;

θ = the angle between the directions of the rope;

W = pull on individual wire, equal to P divided by the number of wires if all wires are of equal diameter.

FIG. 2.

Then
$$R = \frac{d^2}{4\ cos.\ \frac{\theta}{2}} \sqrt{\frac{E}{W}}$$

139.—If R is greater than the radius of the sheaves, $2R$ should be used in place of D in Formulas 1, 2, and 3. The formula is only valid for θ between 110 and 180 degrees.

Strength of Gear Teeth.

140.—The strength of cut gear teeth shall conform to the following formula, one tooth only taking pressure:

$$P = fpb\left(0.154 - \frac{0.912}{n}\right)\frac{600}{600 + V}, \text{ in which}$$

P = pressure on tooth, in pounds;

f = permissible unit stress = 17 000 lb.;

p = pitch, in inches;

b = face or breadth of tooth, in inches;

n = number of teeth in gear;

V = velocity on pitch circle, in feet per minute.

141.—The strength of machine-moulded teeth shall be calculated by the foregoing formula, taking f = 15 000 lb.

142.—The strength of shrouded teeth shall be computed as for uncut teeth, the purpose of the shrouding being to provide for future wear of pinions.

143.—The foregoing formula is for involute teeth having an angle of obliquity equal to 20 degrees.

Pressure on Rollers.

144.—The pressure, in pounds per linear inch, on rollers at rest shall be, for rolled and cast steel, 600 d, where d equals the diameter of the roller, in inches.

Unit Stresses for Bearing on Rotating and Sliding Surfaces.

145.—The maximum bearing values for rotating and sliding surfaces, in pounds per square inch, shall be as follows:

For bearings on which the speed is slow and intermittent:

	Pounds per square inch.
146.—Pivots for swing bridges: Hardened tool steel on special phosphor-bronze	3 000
147.—Trunnion bearings on bascule bridges: Axle steel on phosphor-bronze, average	1 500
and never greater than 1 700 lb. for maximum bearing for any position of the bridge.	
148.—Wedges: Cast steel on cast steel or structural steel	500
149.—Screws which transmit motion on projected area of thread	200
150.—For ordinary cases, parts moving at moderate speeds:	
Hardened steel on hardened steel	2 000
Hardened steel on bronze	1 500
Tool steel (not hardened) on bronze	900
Structural steel on bronze	600
Cast iron on structural steel	400
Cast iron on cast iron	400
On cross-head slides, speed not exceeding 600 ft. per min.	50

151.—In order to prevent heating and seizing at higher speeds, the pressure on pivots or foot-step bearings for vertical shafts and journals shall not exceed:

$$\text{On pivots} \ldots\ldots\ldots p = \frac{40\ 000}{n\ d} \text{ per square inch.}$$

$$\text{On journals} \ldots\ldots p = \frac{300\ 000}{n\ d} \text{ per square inch.}$$

Where n = number of revolutions per minute,
and d = diameter of journal or pivot, in inches.

152.—For crank pins and similar joints with alternating motion, the limiting bearing values given in the above formula may be doubled.

153.—The permissible pressures, in pounds per linear inch of roller in motion, shall be as follows: Stresses on Rollers.

For cast iron	$p = 200d$
For steel castings	$p = 400d$
For axle steel	$p = 500d$
For tool steel	$p = 800d$
For hardened tool steel..................	$p = 1\,000d$

Where p = pressure per linear inch of roller,
and d = diameter of roller, in inches.

154.—The foregoing values are for rollers and bearing surfaces of the same material; if rollers and bearing surfaces are of different materials, the lower value shall be used.

Motors.

155.—The kind of motor best adapted to any particular case depends on local conditions, and should be left to the judgment of the Engineer.

156.—If the bridge is operated by mechanical power, the motor shall be of ample capacity to move or turn the bridge at the required speed. All machinery parts shall be designed with sufficient strength to resist the greatest pressure which can be exerted by the motor. No matter what mechanical power is used, all bridges shall also be provided with hand-power operating machinery. Mechanical Power.

157.—Friction brakes, to be operated by hand or foot, shall be provided where the motor is located in the operator's house. They shall be attached to the secondary shaft of the motors which connect to the moving gear, and shall have sufficient capacity to stop or hold the moving span in any position, under all conditions. Friction Brakes.

158.—If mechanical power of any kind is to be used for operating a movable bridge, a suitable house shall be provided for the operator. The house shall be of such dimensions as required for the purpose for which it is to be used. It shall be placed in a position where the operator can observe the signals and see the approaching vessels and trains, and with enough windows of sufficient size, so that this view will not be obstructed. If the operator's house is above or below the floor of the bridge, suitable steel or iron stairs with railings shall be provided to lead from the floor of the bridge to the floor of the operating house. The house shall be of fire-proof construction, consisting of a steel frame, steel floor-joists and a fire-proof floor. If the house contains motors and machinery, the floor shall preferably consist of steel plates, but, if the motors are located elsewhere, the floor between the joists may be of concrete construction. The sides and roof shall be of metal, concrete or any other non-combustible material. The hand-rail for stairways and other plates shall be of $1\frac{1}{2}$-in. gas pipe. Operator's House.

Heating of Operator's House.

159.—Whenever the climatic conditions require it, provision shall be made for heating the operator's house. If steam power is used, the house shall be heated by a steam coil or radiator fed from the boiler. If electric power is used, the heat may be supplied by electricity. If gasoline is used, or any other power which cannot be utilized for heating, a coal, wood, petroleum, or gas stove, as directed by the Engineer, shall be provided.

Steam Engine.

160.—If a steam engine is used, it shall consist of a double-cylinder, reversing engine, the piston speed of which shall not exceed 200 ft. per min.; it shall develop the desired power and speed with a steam pressure of 50 lb. per sq. in. The engine shall be connected to the operating machinery by an approved friction clutch, arranged so that the moving and locking machinery can be operated alternately or stopped without stopping the engine.

Steam Separator.

161.—In the steam supply pipe, and close to the steam chest, shall be placed a steam separator. This separator, under test with quality of steam as low as 66%, shall show an average efficiency of 85% in five tests.

Boilers.

162.—The steam shall be generated by one or two upright, tubular boilers, each of which shall have twice the capacity of the engine. The boilers shall be designed for a steam pressure of 150 lb. per sq. in., and shall be adapted to the kind of fuel specified by the Engineer; they shall be of open-hearth steel in accordance with the specifications for boiler plates, Paragraphs 246 to 251, inclusive. They shall be encased in asbestos and covered with Russia iron.

163.—The boilers shall also be in accordance with the specifications of the Mechanical Department of the Railway Company, and shall conform to the civil laws.

Flues of Boilers.

164.—Vertical boilers shall have submerged flues at the top.

Horse-Power of Boilers.

165.—The total horse-power of the boilers shall be twice that of the engine, and shall be computed by the following rule: Calculate the inside area of the tubes, the area of tube sheet next to the fire, and the sides of the fire-box where this is in contact with the fire. Take the sum of these areas in square feet and divide by fifteen. The intention is to allow 15 sq. ft. of heating surface per horse-power. At least $\frac{1}{3}$ sq. ft. of grate surface shall be provided per horse-power.

Equipment of Engine-Room.

166.—The engine-room shall be provided with a steel water tank of sufficient capacity; a duplex, steam feed-pump; and an injector for each boiler, with necessary pipes and connections for feeding boilers separately or together; steam water-lifters with necessary strainers, flexible hose, and piping to lift the water from the river into the tank; a coal hoist and a steel coal-bin of sufficient capacity. The engine-room shall be provided with a suitable indicator for recording the positions of the moving span in turning and locking. A work-bench with

a full set of machinist's tools shall be provided, such as a vise, wrenches, chisels, hammers, files, oilers, oil-cans, and oil-tank.

167.—A whistle having a bell 5 in. in diameter and 12 in. long, shall be installed complete. If operated by air, the compressor and air tank shall conform to the following specifications: The compressor shall be motor driven, the motor and compressor being on one frame, and geared. All working parts shall be completely enclosed, and self-lubricating. The compressor shall have a piston displacement of from 25 to 30 cu. ft. per min. when working against a tank pressure of 90 lb. per sq. in. The compressor shall be provided with strainer, and automatic governor and switch, in order that the compressor may start and stop automatically at any predetermined tank pressure. The air receiving tank shall be 36 in. in diameter and 8 ft. long, or of equal capacity. The tank shall be galvanized, and good for a working pressure of 100 lb. per sq. in. It shall be provided with pressure gauge and pigtail, pop-valves and drain cock, and have standard flanges bushed for 1½-in. pipe. The Contrator shall furnish all pipe, pipe fittings, and valves, and all shall withstand a working pressure of 100 lb. per sq. in. Whistle.

168.—If a gasoline motor or other internal-combustion motor is used, a low-speed engine of the most substantial kind shall be selected, the maximum piston speed of which shall not exceed 350 ft. per min. The engine shall have a reversing gear provided with approved friction clutches, to be operated by a hand-wheel. The countershaft connecting the engine with the operating machinery shall be provided with disengaging couplings, arranged so that the moving and locking machinery can be operated alternately and in either direction without stopping the engine. Motors of 10 h.p. and more shall be started by compressed air. The engine-room shall be provided with a water tank of sufficient capacity. The gasoline tank shall be located outside of the engine-house. The engine-room shall be provided with indicators for recording the positions of the moving span, and lifting and locking apparatus. A work-bench with a full set of machinist's tools, etc., shall be provided, the same as specified for steam engines. Gasoline Motor.

169.—Electric motors and generators, if for direct current, shall be of the railway series, interpole type, water-proof, with slotted-drum armature, and form-wound armature coils. They shall be a standard commercial type in common use. Electric Motors.

170.—The coils shall be impregnated.

171.—Motors, generators, automatic circuit breakers, solenoids, brakes, and other electric mechanism shall be tested at the factory by the manufacturer in the presence of the Railway Company's inspector.

172.—The rating of a direct-current motor is the horse-power output at the armature shaft which gives a rise of temperature above the surrounding air (referred to a room temperature of 25° cent.) not exceeding Testing of Motor.

90° cent. at the commutator and 75° cent. at any other part after one hour's continuous run at its rated voltage (and frequency in the case of an alternating-current motor) on a stand with the motor covers removed and with natural ventilation. The rise in temperature is to be determined by thermometer, but the resistance of no electric circuit in the motor shall increase more than 40% during the test.

173.—Direct-current motors shall be capable of carrying a load of 200% for 3 min. with the same temperature rise and momentarily of 400% without injury, starting cold in each instance.

Torque of Motor.

174.—The motors under test shall develop the required horse-power and torque at the armature shaft. Characteristic curves showing the results of the test shall be furnished by the manufacturer.

175.—The motor frame shall have two bearings for the countershaft and shall have a forged-steel cut pinion, out of one piece, keyed to the end of the armature shaft and secured by a lock-nut.

175a.—If the motor is enclosed in a case, as mill motors are, small openings of sufficient size shall be provided in the case for the inspection, removal, and replacing of brushes.

176.—One cast-steel cut gear, bored and key-seated for attachment to the countershaft, shall be furnished with the motor. The gear and pinion shall be covered by a sheet-steel or malleable-iron split gear case, supported by the motor frame and completely covering the gear and pinion. An opening, with a hinged cover, shall be provided in the gear case for inspection and oiling. The gear ratio shall be such that the full speed of the countershaft will not be more than 125 rev. per min.

Spare Motor Parts.

177.—For each size of motor furnished, the Contractor shall supply the following spare parts: One armature, one field coil, one pinion, one gear, and one set of brushes. These parts shall be finished and fitted in such a manner as to admit of being installed in their respective places without further fitting or adjustment.

Mounting Motors.

178.—The motors shall be mounted in such a manner as to admit of easy access for inspection and repairs; they shall be supported securely by brackets or suitable foundations.

179.—If the machinery and motors are on the moving span, they shall be capable of being operated satisfactorily in any position of the span.

Controllers.

180.—The controllers for motors shall be located in the operating-house. The controllers shall be of the reversing drum type, with magnetic blow-out, and shall be capable of varying and maintaining the speed of the motors throughout the entire range desired, without injurious sparking, and without shock due to sudden variation in speed. The controllers shall be capable of doing their work for the usual loads, and excess loads, that may come upon the motors, with a temperature rise not exceeding that specified for the motors.

181.—One controller with the necessary resistances shall be furnished for controlling the operation of each main operating motor. They shall be connected so that the motors may be operated together.

Type of Controllers. 182.—The controllers shall be of the series-parallel type; or of the type in which the field is varied, as may be done for the interpole type of motor.

Controllers, Where Needed. 183.—One controller for direct-current motors shall be furnished for the operation of the rail locks, and one for bridge locks. These controllers shall be designed so that the operation of any motor can be cut out by pulling a switch on the switch-board, without affecting the operation of any of the other motors.

Automatic Cut-Offs. 184.—An automatic cut-off or short-circuiting device shall be provided which will throw out the circuit breakers, cut off the current from the operating motors and set their brakes when the bridge is 5° from its open position, and its closed position. Spring switches shall be provided which, if closed and held closed, will put the cut-offs out of commission and thus enable the bridge tender to fully close or open the bridge.

185.—The end lock motor shall be stopped and its brake set automatically at each end of its travel.

Resistances. 186.—Resistances shall be of the cast-grid type, and of such capacity that the motor can be operated continuously at any point of the controller when developing full-load torque, or for 10 min. when developing 50% over-load torque, without sufficient rise in temperature of the resistance to cause deterioration of any part. The resistances shall be mounted so as to admit of free ventilation and be without injurious vibration.

Electric Brakes. 187.—The main operating motors, rail lock motors, and bridge lock motors shall be provided with approved post brakes which are held in set position by a spring with such force as to overcome not less than 50% of the maximum torque required. The friction surfaces are to be of materials not affected by moisture. The brakes are to be released by solenoids of ample power and heating capacity whenever the motors are taking current, and are to be automatically set whenever the current fails or is cut off from the motors. Weather-proof motors shall be provided with weather-proof solenoids. Brakes shall be provided with a foot-switch release for coasting purposes. Means shall be provided for mechanically releasing the brakes when the bridge is to be operated by hand or other equipment.

Emergency Brakes. 188.—An additional emergency brake shall be provided and applied to the main operating machinery. This shall be released by means of a motor-operated mechanism furnished by the Eectrical Contractor, which shall hold the brake in release as long as the current is applied to the brake motor. Cutting off the current from this brake motor, or any failure of current, will result in the instantaneous application of the

brake. This brake will be normally set, but will be released by the operator before starting the bridge, and be held in release during the entire operation unless an emergency condition arises requiring brake power in excess of that offered by the motor brakes, in which case it may be instantly applied by the operator. After the bridge has been closed and traffic has been resumed, this brake will again be applied. This portion of the equipment shall be designed so that it will not be injured if left in release indefinitely. Proper means shall be provided for releasing the brake mechanically when the bridge is to be operated by hand or emergency-power equipment.

189.—The emergency brake motor circuit is to be independent of the general interlocking system, and there shall be a mechanical interlocking device which will prevent the main leaf motors and the emergency brake from being used one against the other.

190.—The emergency brake switch shall be attached to the controller stand within easy reach of the operator and proper labels shall be placed back of the switch handle to indicate "Set" and "Released" positions of the brake.

Current Supply. 191.—Unless the current supply is taken from more than one source, it shall be conducted to the switch-board in two independent conductors, one for the supply, and one for the return current.

Submarine Cables. 192.—Submarine cables, if needed, will be furnished and laid by the Railway Company.

193.—The wiring from the collector rings for the electrical equipment of the bridge shall be furnished by the Contractor.

Qualities of Wire and Insulation. 194.—The quality of all wires and insulation shall conform to the specifications of the Railway Signal Association, as revised and adopted in October, 1911, and contained in Volume 8 of the *Proceedings* of that Association.

Conduits and Minimum Size of Wire. 195.—If wires are to be placed in conduits, the conduits shall be of ample size, sherardized, and loricated on the inside. No wire less than No. 12, B. & S. gauge, shall be used.

Condulets and Factory Ells. 196.—Conduits shall be of sufficient size to allow the wires to be easily drawn in. No joints are to be made inside of a conduit. Condulets and factory ells shall be used. Condulets, ells, and conduits shall be sherardized, and loricated inside.

Wiring, etc., to Conform to Codes. 197.—The wiring, motor installation, and the whole electric equipment must conform to the underwriter's code, and to the city code, if the bridge is subject to city authority.

Fuses. 198.—Enclosed fuses shall be used.

Minimum Stranded Wire. 199.—No wire smaller than No. 10, B. & S. gauge, stranded wire shall be used.

Wires to be Tagged. 200.—Wires when installed shall be permanently tagged and numbered so that any wire can be traced from the switch-board to the motors, and to the source of power.

201.—Ground connections of ample area shall be provided. **Ground Connections.**

202.—A switch, of the quick-break type, shall be provided for each supply wire. Each motor circuit and each light, signal, indicator, or other circuit shall be provided with switches which are approved by the Railway Company's Engineer. The switches shall be mounted on an enameled slate panel switch-board (not less than 1¼ in. thick, and free from metallic veins, or flaws) in the operator's house. The switch-board shall be large enough to carry the meters, switches, cut-outs, fuses, etc. Switches, cut-outs, buttons, etc., shall be provided with plates designating their use. **Quick Brake Switch and Switch-Board.**

203.—An automatic circuit breaker shall be placed on the switch-board in the operating motor circuit of the bridge. Each line to the motor, each line to the electric brakes, and each lighting, signal, indicator, or other circuit, shall be protected by enclosed fuses. **Automatic Circuit Breaker.**

204.—Any circuit whatsoever shall be protected by fuses, circuit breakers, or equivalent devices, which will insure the excessive current being cut off before any parts are damaged.

205.—The feeders shall be protected by a pole-switch fuse and lightning arrester mounted on a non-combustible and non-absorbent insulating base. **Lightning Arrester.**

206.—Lightning arresters shall be placed as near as practicable to the parts to be protected, and away from combustible material. A No. 4, B. & S. gauge, wire should be used for the connection; this wire should run in a straight line to a ground plate, and not be connected to any structural parts. To avoid inductive resistances, the wire should not run through a conduit. If a choke-coil is used, it should be thoroughly insulated from the ground and other conductors.

207.—The connections of parts in contact with track shall be such as to allow no short circuiting of track signals. **Short Circuiting.**

208.—Electric contacts shall be protected from the weather or accumulations of dirt. **Protection of Electric Contacts.**

209.—Motors must be housed in weather-proof metal housing. This housing must be large enough to allow the inspection and oiling of the motor. It must be readily removable so that access to the motor may be obtained. No metal in this housing shall be less than No. 16, U. S. Standard, gauge; it shall be galvanized. **Housing of Motors.**

210.—Solenoids and electrically-operated brakes shall be housed. **Housing of Solenoids, etc.**

211.—The Contractor shall provide and install electric light indicators for the purpose of showing the operator the various positions of the bridge, especially the fully open, entirely closed, nearly open, and nearly closed positions of the bridge, and the fully open and fully closed positions of the rail lock and bridge locks. **Indicator Lights.**

212.—A volt meter, ammeter, and watt meter shall be provided on the switch-board. The use of external multiple shunts will not be permitted. **Volt Meter, etc.**

Ground Detector. 213.—The switch-board shall be furnished with one 2-c. p. lamp for detecting ground, and a 2-c. p. lamp for illuminating the ammeter and volt meter scales.

Lamps for Lighting. 214.—In the operator's house shall be placed ten 16-c. p. lights, and additional lights about the machinery and such other lights as the Engineer may direct. For all lights in the house above ten in number, the Railway Company will pay the regular market price or furnish them to the Contractor.

215.—Lights of 16-c. p. shall be placed outside at the head and foot of stairways or similar paths. All lights in the house shall have tungsten filaments.

Channel Lights. 216.—The Contractor shall furnish warning and channel lights and signals, in accordance with the U. S. Government requirements, or other harbor requirements.

Alternating Current Motors. 217.—Alternating motors shall be of the three-phase induction type with slip-rings, rotor-wound, 25 cycles and 220 voltage, unless otherwise specified. The resistances for varying the speed shall be in series with the rotor circuit, and shall be such as to affect evenly all three phases. Motors of 5 h.p. or less may be of the squirrel-cage type.

218.—The methods of testing outlined for the direct-current motors shall apply to the alternating motor.

Control of Motors. 219.—The control of motors shall be electrically interlocked with each other and with the signal system, and the bridge shall be controlled in such a way that the end locks or wedges cannot be released until the signals have gone to the danger position and the derails are set, or the bridge motor started until the end locks and wedges have actually been released. In closing the bridge, the control shall be such as to make it impossible for the operator to move the end locks or wedges until the bridge has been completely closed or to set the signals at safety until the bridge has been closed and the end locks and wedges are in place.

Railway Signal System. 220.—The company will furnish and install the railway signal system, also the master lever and all necessary devices controlling the interlock between this signal system and the bridge as a whole. The Contractor shall furnish and install the necessary devices for interlocking the various parts of the bridge with each other and for connection to the Company's master lever.

Specifications for Special Metals Used for Machinery Parts.

Qualities of Machinery Steel. 221.—Steel for castings may be made by the open-hearth or crucible process.

222.—All castings shall be annealed unless otherwise specified.

223.—Phosphorus	....	0.05% maximum.
Sulphur	....	0.05% maximum.

224.—Minimum physical qualities, as determined on a standard test specimen, of ½ in. diameter and 2 in. gauged length:

Tensile strength, in pounds per square inch......	70 000
Elongation: percentage in 2 in................	18
Contraction of area: percentage...............	25

225.—A test to destruction may be substituted for the tensile test, in the case of small or unimportant castings, by selecting three castings from a lot. This test shall show the material to be ductile, free from injurious defects, and suitable for the purpose intended. A lot shall consist of all castings from the same melt or blow, annealed in the same furnace charge.

226.—Castings shall be true to pattern, free from blemishes, flaws, or shrinkage cracks. When the bearing surface of any steel casting is finished there shall be no blow-holes visible, exceeding 1 in. in any direction, nor exceeding ½ sq. in. in area. The length of blow-holes cut by any straight line laid in any direction shall never exceed 1 in. in any 1 ft. **Flaws in Castings.**

227.—No blow-hole exceeding one-half the above dimension and area will be allowed in any gear tooth, or in the rim at the root of the teeth. **Blow-Holes in Gear Wheels.**

228.—The correction of defects in castings, by welding electrically by thermit or by similar processes, will not be allowed. **Electric Welding.**

229.—Large castings shall be suspended and hammered all over. No cracks, flaws, defects, or weakness shall appear after such treatment. **Testing of Large Castings.**

230.—A specimen (1 in. by ½ in.) shall bend, cold, around a diameter of 1 in., through an angle of 90°, without fracture on the outside of the bent portion.

231.—The number of standard test specimens shall depend on the character and importance of the castings. A test piece shall be cut, cold, from a coupon to be moulded and cast on some portion of one or more castings from each melt or blow, or from the sink-heads (in case heads of sufficient size are used). The coupon or sink-head must receive the same treatment as the casting or castings, before the specimen is cut out, and before the coupon or sink-head is removed from the casting.

232.—Turnings from the tensile specimen, or drillings from the bending specimen, or drillings from the small test ingot, if preferred by the inspector, shall be used to determine whether or not the steel is within the limits in phosphorus and sulphur specified in Paragraph 223 concerning chemical properties.

Steel Forgings.

233.—Steel forgings may be made by the open-hearth or crucible process. **Qualities of Steel Forgings.**

234.—Phosphorus	0.04% maximum.
Sulphur	0.05% maximum.

235.—Minimum physical properties as determined on a standard turned test specimen of ½ in. diameter and 2 in. gauged length:

Tensile strength, in pounds per square inch,	55 000 to 65 000
Elongation: percentage in 2 in.	28

236.—A specimen (1 in. by ½ in.) shall bend, cold, 180°, around a diameter of ½ in., without fracture on the outside of the bent portion. The bending may be effected by pressure or by blows.

237.—The number and location of the test specimens to be taken from a melt, blow, or forging shall depend on their character and importance, and, therefore, must be regulated by individual cases. The test specimen shall be cut, cold, from the forging, or full-sized prolongation of the same, parallel to the axis of the forging and half way between the center and the outside; the specimens shall be longitudinal, *i. e.,* the length of the specimen shall correspond with the direction in which the metal is most drawn out or worked. When forgings have large ends or collars, the test specimens shall be taken from a prolongation of the same diameter or section as that of the forging back of the large end or collar. In the case of hollow shafting, either forged or bored, the specimen shall be taken within the finished section prolonged, half way between the inner and outer surfaces of the wall of the forging.

238.—Turnings from the tensile specimen, or drillings from the bending specimen, or drillings from the small test ingot, if preferred by the inspector, shall be used to determine whether or not the steel is within the limits in chemical composition specified in Paragraph 234.

239.—Forgings shall be free from cracks, flaws, seams, or other injurious imperfections, and shall conform to the dimensions shown on the drawings furnished by the purchaser, and shall be made and finished in a workmanlike manner.

240.—All forgings shall be annealed.

Axle Steel.

Qualities of Axle Steel.

241.—Axle steel may be made by the open-hearth or crucible process.

242.—Phosphorus	0.05%	maximum.
Sulphur	0.05%	maximum.

243.—Minimum physical properties, as determined on a standard turned test specimen of ½ in. diameter and 2 in. gauged length:

Tensile strength, in pounds per square inch	80 000
Elongation: percentage in 2 in.	20

244.—A specimen (1 in. by ½ in.) shall bend, cold, 180°, around a diameter of 1½ in., without fracture on the outside of the bent portion. The bending tests may be made by pressure or by blows.

245.—Turnings from the tensile test specimen, or drillings from the small test ingot, if preferred by the inspector, shall be used to determine whether the melt is within the limits in chemical composition specified in Paragraph 242.

Boiler Plates.

Qualities of Boiler Plate Steel.

246.—The steel used for boilers and fire-boxes shall be made by the open-hearth process.

247.—Phosphorus	0.04%	maximum.
Sulphur	0.04%	maximum.

248.—The physical properties required shall be as follows:

Tensile strength desired, in pounds per square inch, 60 000.

Elongation : minimum percentage in 8 in. $= \dfrac{1\ 500\ 000}{\text{Ultimate strength.}}$

Character of fracture	Silky.
Cold bends, without fracture	180° flat.

249.—The ultimate strength shall come within 4 000 lb. of that desired.

250.—Chemical determinations of the percentage of carbon, phosphorus, sulphur, and manganese, shall be made by the manufacturer from a test ingot taken at the time of the pouring of each melt of steel, and a correct copy of such analysis shall be furnished to the Engineer or his inspector. A check analysis shall be made from the finished material, if called for by the purchaser, in which case an excess of 25% above the required limits will be allowed.

251.—Specimens for tensile and bending tests for plates shall be made by cutting coupons from the finished product, which shall have both faces rolled, and both edges milled to the usual form of the standard test specimen, 1½ in. wide on a gauged length of at least 9 in.; or with both edges parallel.

Nickel Steel for Machine Parts.

Qualities of Nickel Steel.

252.—Nickel steel shall be made by the open-hearth process.

	Plates, shapes and bars.	Rivets.
253.—Phosphorus shall not exceed	0.04%	0.04%
Sulphur " " "	0.05%	0.04%
Nickel, not less than	3.00%	3.25%

254.—The physical properties required shall be as follows:

	Plates, shapes, bars, and forgings, pounds per square inch. Minimum.	Rivets.
Tensile strength	80 000	60 000 to 70 000
Elastic limit	50 000	40 000 minimum

Elongation, percentage in 8 in., for plates, shapes, bars, and forgings ; and also for rivets $= \dfrac{1\,600\,000}{\text{Ultimate strength}} =$ minimum.

Elongation, percentage in 2 in., for forgings = 25.

255.—Specimens cut from forgings (1 in. by ½ in.) shall bend, cold, 180°, around a diameter of 1 in., without fracture on the outside of the bent portion.

256.—Specimens cut from plates, shapes, and bars shall bend, cold, 180°, around a diameter of three times their thickness, without fracture on the outside of the bent portion.

257.—Each rivet rod shall bend 180°, flat, on itself, without fracture on the outside of the bent portion.

258.—Rivet rods shall be tested as rolled.

259.—The fracture of all tension tests shall show a fine silky texture, of a uniform bluish gray or dove color, free from black or brilliant specks, and shall show no sign of crystallization.

260.—All nickel-steel forgings shall be properly annealed.

261.—Annealed eye-bars and similar members, when full-sized pieces are tested, shall comply with the following requirements:

Minimum ultimate tensile strength, in pounds per square inch	75 000
Minimum elastic limit, in pounds per square inch.	45 000
Minimum elongation in 10 ft., including fracture.	12%
The fracture shall be mostly silky, and free from crystals.	
Full-sized pieces shall bend, cold, 180°, around a diameter of twice their thickness, without fracture.	

Tool Steel.

Qualities of Tool Steel.

262.—This steel is generally used for parts which require hardening or oil tempering, such as pivots, friction rollers, ball-bearings, and springs.

263.—Tool steel shall be made by the open-hearth or crucible process.

264.—Carbon	1.00%	minimum.
Phosphorus	0.04%	maximum.
Sulphur	0.04%	"
Manganese	0.50%	"

Phosphor-Bronze.

Qualities of Phosphor-Bronze.

265.—Special phosphor-bronze shall be used for high pressures and slow speed.

266.—The metal shall have a minimum elastic limit in compression of 27 000 lb. per sq. in. The permanent set at 100 000 shall not exceed $\frac{1}{10}$ in.

267.—A test piece shall be cut from a coupon to be moulded and cast on some portion of each casting. Test pieces shall be 1-in. cubes, finished.

268.—Phosphor-bronze composed of the following ingredients and of the following proportions has given satisfactory results:

Copper	79.7 per cent.
Tin	10. " "
Lead	9.5 " "
Phosphorus	0.8 " "

Babbitt Metal.

269.—Babbitt metal composed of the following ingredients and of the following proportions has given satisfactory results and a low coefficient of friction (0.03 to 0.04): **Qualities of Babbitt Metal.**

Copper	3.6 per cent.
Tin	89.3 " "
Antimony	7.1 " "

270.—It is the purpose of these specifications to provide a first-class structure. They are intended as an aid in designing and fabrication. The subject of machine design is so great and varied that no single work of this character can cover all points. As a further aid in securing a first-class structure, the following works will be considered authoritative in the order named: **Purpose of the Specifications.**

1. Unwin's Machine Design, Part I, Ed. 1909.
   Unwin's Machine Design, Part II, Ed. 1902.
2. A Manual of Machine Design, etc., by Low and Bevis, 11th Impression.
3. Reuleaux's Constructor, Translated by Suplee.
4. Kent's Pocket Book, 8th Ed.

271.—Machine parts shall be designed, if practicable, by the methods of applied mechanics, but such designs shall be viewed in the light of experience. It should be borne in mind that machine design is not based on the precise methods in vogue for statical structures.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

THEORY OF REINFORCED CONCRETE JOISTS.*

BY JOHN L. HALL, M. AM. SOC. C. E.

In computing the strength of reinforced concrete floor slabs, it is usual to disregard the tensile resistance of the concrete. That portion of the concrete on the tensile side of the neutral plane is considered only useful for covering the steel, for helping to resist shear, and for forming a ceiling. A small part of this concrete would ordinarily be sufficient to cover the steel and furnish the necessary resistance to shear. The remainder is a heavy and somewhat expensive material for a ceiling. Particularly is this true in the case of long spans which require thick slabs.

By keeping the reinforced steel in large units, a series of parallel concrete joists may be formed, instead of a flat slab. With a thin slab over the top, lightly reinforced transversely, the joists become a system of small T-beams. The expensive form work of such a system is one objection to it, and the preference for a flat ceiling is another. To obviate these objections, burned clay hollow tile with plaster ceiling, or sheet-metal tile with metal lath and plaster ceiling, has been used.

The purpose of this paper is not to discuss the relative merit or economy of these several methods of construction, but rather to discuss the things which should be considered in computing the strength of such a system of joists. Various claims are made as to the work performed by clay tile in combination with concrete joists. It is not

*This paper will not be presented at any meeting, but written communications on the subject are invited for publication with it in *Transactions*.

the intention to discuss this matter at present. The spaces between joists, therefore, will be assumed to be voids. Fig. 1 shows how such a system was used in a recent design. Fig. 2 is a section through the joists, and Fig. 3 is a section through the beams. Fig. 4, illustrating the mode of bending in a beam with fixed ends, is introduced for the purpose of reference in what follows.

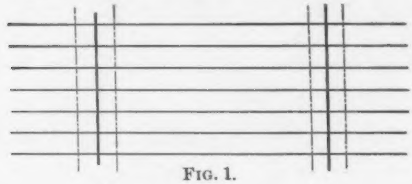

FIG. 1.

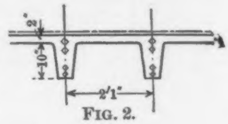

FIG. 2.

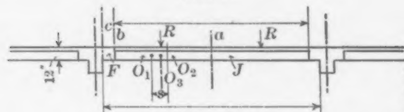

FIG. 3.

21′8″ Span
Moment at a (Fig. 3) = 4540 ft.-lb.
Reaction R = 1640 lb.
Moment at b = −7900 ft.-lb.
" " c = −11000 " "
23′10″ Span
Moment at a = 5400 ft.-lb.
Reaction R = 1790 lbs.
Moment at b = −9100 ft.-lb.
" " c = −13700 " "

Round Rods are indicated thus (ϕ)
Square, Cold-twisted Bars are indicated thus (♦)

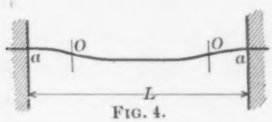

FIG. 4.

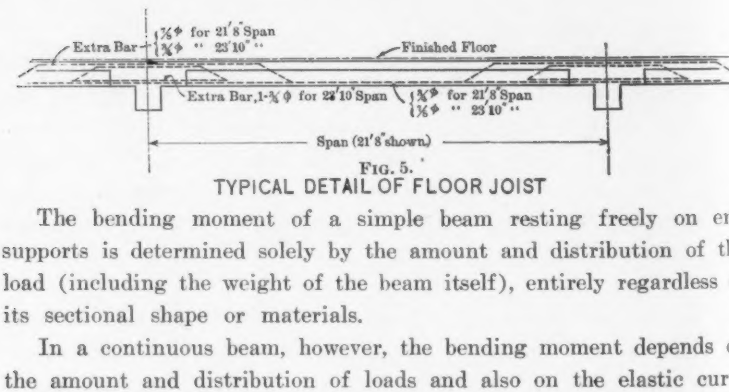

FIG. 5.
TYPICAL DETAIL OF FLOOR JOIST

The bending moment of a simple beam resting freely on end supports is determined solely by the amount and distribution of the load (including the weight of the beam itself), entirely regardless of its sectional shape or materials.

In a continuous beam, however, the bending moment depends on the amount and distribution of loads and also on the elastic curve or deflection of the beam. The elastic curve is influenced by the shape and composition of the beam. What the actual bending moments are in reinforced concrete, therefore, is not known. It is a matter of much difference of opinion among engineers, as shown

by the discussion before this Society following the Progress Report of the Special Committee on Concrete and Reinforced Concrete.

As commonly given in books on mechanics, the bending moments and reactions for continuous beams are calculated for the ideal case of homogeneous beams of uniform sections resting freely on level supports evenly spaced. Such cases do not occur in building construction. After many tests of actual construction, the formulas given in the building laws, although based on the ideal case, are recognized as safe, and are in general use; yet it is well to remember always that these formulas are only approximations. The nearer a design approaches the ideal case referred to, the more nearly do the formulas approach the truth. Any material deviation in design from the ordinary approximation to the ideal condition presents a case for special study and determination.

It is thought that the laws of deflection and the amount of deflection under working loads are much the same in reinforced and in plain concrete. Certainly, the small deflections in reinforced concrete, as compared with those in structural steel, show that in the former the concrete is more of a controlling factor in deflections than the steel. This theory is consistent also with calculated deflections using the moment of inertia of sections containing usual percentages of steel. It would seem, therefore, that the reinforced steel has very little influence on the elastic curve of the kind of construction under investigation, except in so far as it prevents tensional rupture and thereby permits higher stresses. In this view of the subject it is apparent that the reinforcing steel should be placed so as to resist rupture where it would be most likely to occur in the concrete.

Looking at the floor construction of Figs. 1, 2, and 3, if it be assumed that this construction is essentially a continuous flat slab, and that the maximum bending moment is at the center line of supporting beams, then the maximum stress both in compression and tension will be along this center line, and the stresses will diminish, according to some law, to zero at the line of contraflexure. Suppose, now, that a large part of the concrete on the compression side of the slab be removed at some place between the center line of the supporting beam and the line of contraflexure; a plane of weakness is introduced which may cause failure where the moment is considerably less than the maximum. This is inconsistent with the

premises, and shows that the width of the 12-in. flanges of the beams must be considered in calculating the strength of the floor slab so-called.

If it is assumed, however, that the construction consists of a series of joists or small T-beams with one or both ends fixed, then the danger section of a joist occurs at a fixed end, where it joins a beam, and the analysis becomes straightforward and consistent. It is thought, therefore, that the joists should be designed for fixed ends, in accordance with actual conditions.

An objection which might be made to this procedure is that the beam flanges, into which the ends of the joists are said to be fixed, are themselves capable of deflection, so that the ends of the joists are inclined slightly instead of level. This deflection at the edge of the flanges, however, must necessarily be very small, being estimated according to the respective moments of inertia, and is only 40% of what it would be at same line if the joist section ran without change to the center of the beams.

The effect of this slight inclination of the supports would be to increase slightly the positive moment at the center of the joists and to reduce slightly the negative moment at the fixed ends. This reduction of negative moment would cause the calculated negative moment to err on the side of safety, and the slightly increased positive center moment would utilize more economically the excess strength provided at the center.

The negative bending moment along the center line of the beams is the sum of the moments of the distributed loads out to the line of contraflexure, and of the concentrated loads along the latter line. Any change in design that tends to move the line of contraflexure away from the supports and toward the center of the span, would tend to increase the negative moment at the center of the supports. It is conceivable, therefore, that the negative moment in the case under discussion might be somewhat more than $\frac{W\,L}{12}$. If so, additional steel should be provided in the top of the slab across the beams.

The formula, $\frac{W\,L}{12}$, expresses the bending moment at either end of a beam of constant section uniformly loaded and having fixed ends. The points of contraflexure, O (Fig. 4), are located $0.211L$ from the

ends. The central portion, O-O, may be considered as a simple beam, uniformly loaded. The end portion, a-O, may be considered as a cantilever uniformly loaded from a to O and supporting at O the reaction from O-O. From these conditions the shear and bending moment can be readily computed for any section in the length of the beam.

In similar manner, a continuous beam is restrained by bending moments at the supports. The points of contraflexure, however, from which the moments may be computed, are not located as easily. Their position is affected by the number and the relative length of the spans and by the distribution of the live loads, whether on some or all of the spans, and by other considerations. In order to simplify the computations, the building laws authorize the use of the formula, $\frac{W\,L}{12}$, for interior spans, and, $\frac{W\,L}{10}$, for end spans. This method of calculating is only approximately correct. Its error is usually, although not always, on the side of safety. The formula for interior spans is the same as that for beams with both ends fixed. For end spans, the formula indicates one fixed end and one partly fixed.

When applicable, the theorem of three moments permits the accurate determination of moments and shears for actual conditions; and, if all the conditions are actually considered, it affords a more scientific method of calculation than the approximate formulas previously stated. The building law recognizes the validity of scientific analysis, and caution would seem to require such analysis, if attainable, whenever the design varies materially from ordinary conditions of continuity, as, for example, when the spans are very unequal in length, or when the conditions of constant section and free support are deviated from in any marked degree.

The features of this design, which vary from usual conditions of continuity, are: (1) the massive character of the supporting beams; and (2) the sudden change in section where the joists join the flanges of the beams.

The joists in interior spans were assumed to have fixed ends, and an attempt was made to determine the location of the points of contraflexure. Investigations by F. E. Turneaure, Assoc. M. Am. Soc. C. E., indicate that, within ordinary working stresses and percentages of steel, the elastic curve of a concrete beam is not greatly influenced

by the position of the reinforcement. It appears to be desirable, therefore, to ascertain the natural elastic curve of the concrete joists and place the steel where it is needed in conformity thereto.

Two assumptions were tested: (1) That the ends of the joists are fixed at the face of the beam web; (2) that they are fixed at the edge of the beam flanges. The location of the points, *O*, was calculated for both conditions, on the basis of a constant section.

The position, O_1, Fig. 3, obtained by the first calculation would be correct if the flange, *F*, were as flexible as the joist, *J*, while the position, O_2, would be correct if the flange, *F*, were perfectly rigid. As the flexibility of *F* is intermediate between these assumptions, the true position, O_3, must lie between O_1 and O_2, and closer to the one derived from that assumption, which is nearer the truth.

A beam of the section, *F*, it was estimated, would deflect four-tenths as much under a given load and span as one of the section *J*. It is then more nearly a rigid beam than one of the same flexibility as *J*. The point, O_3, therefore, lies nearer O_2 and is four-tenths of the way from O_2 toward O_1.

Having fixed the position of the points, *O*, the remaining calculations are very simple. The typical detail of the joist, Fig. 5, is designed in accordance with the actual conditions as understood, and the necessary resistance is provided. The moments and shears are stated on that figure.

In view of the fact that the moment at the center of the span of a beam with fixed ends is only one-half of $\frac{W\,L}{12}$, it may be asked why this formula is required by law generally for the center of continuous interior spans, and whether such requirement would be justified in the present instance. Analysis of the spans under consideration by the ordinary theory of continuous beams showed that if only one span be fully loaded, the other spans having dead load only, the fully loaded span will have a moment at the center about two-thirds of $\frac{W\,L}{12}$. If the live load on one span only were greatly increased over the dead load, the moment at the center of the span would be increased in a larger ratio; but, in the absence of those conditions, there appears to be nothing gained by increasing the reinforcement at the center of the span.

Again, the theory of continuous beams does not take into account the torsional stiffness of the supporting beams, which, in the present case, is very considerable. Manifestly, the more nearly we approach the ideal condition of fixed end supports for the joists, the less influence will be exerted by conditions outside the particular span considered, so that the probable maximum moment at the center of the span of the joists will always be less than two-thirds of $\frac{W\,L}{12}$, in this particular design.

In the detail herewith submitted, Fig. 6, the reinforcing steel is disposed in accordance with the foregoing statement of theory.

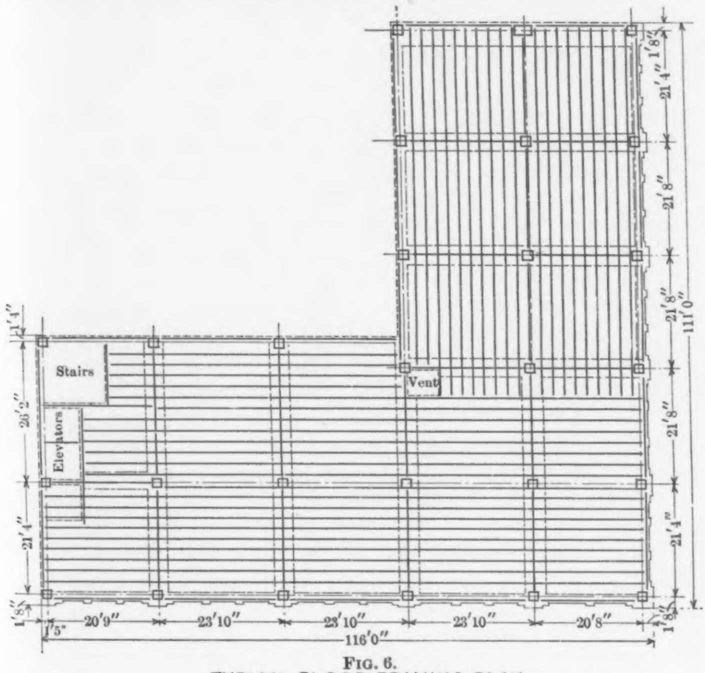

FIG. 6.
TYPICAL FLOOR FRAMING PLAN

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

NOTES ON BRIDGEWORK.
Discussion.*

By William P. Parker, M. Am. Soc. C. E.

William P. Parker, M. Am. Soc. C. E. (by letter).—The author's treatment is on the assumption that a continuous beam on three points of support can be dealt with as two separate beams fixed at the middle point and simply supported at the ends. Mr. Parker.

This assumption is incorrect, as a load on either span affects the reactions on all three supports.

For any load, W (Fig. 4), the sum of the reactions at 1 and 2 is greater than W, while the reaction at 3 is negative. The three reactions, $R_1 + R_2 + R_3$, of course, are equal to W.

The purpose of the investigation seems to be primarily to find the reactions at R_2, the intermediate support, for any load or series of loads. By the method in the paper, the results give a much less reaction than the correct one. The writer, for his work in reinforced concrete design, uses the curves in Fig. 4, which give directly the reaction at all three points of support for a concentrated load in any position and the results from a series of loads can be combined arithmetically to give the resultant reactions. With the reactions found, it is easy to ascertain the bending moment and shear at any point on the beam, and locate points of contraflexure.

Using the notation on Fig. 4: From the theorem of three moments, for any load in the span, 1-2, W produces:

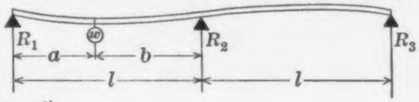
$$\begin{aligned} \text{Moment } M_1 &= \quad 0 \\ \text{Moment } M_2 &= -\tfrac{1}{4}\, wl\,(a-a^3) \\ \text{Moment } M_3 &= \quad 0 \end{aligned}$$

Taking the center of moments at the different supports, and expressing the given moment, the algebraic sum of the reaction and loads

* Continued from August, 1912, *Proceedings*.

Mr. Parker.

CONTINUOUS GIRDER OVER 3 POINTS OF SUPPORT



$M_2 = -\frac{1}{4}\, wl\,(a - a^3)$

$R_1 = +wb - \frac{1}{4} w (a - a^3)$, $R_2 = +wb + \frac{1}{2} w\ (a - a^3)$, $R_3 = -\frac{1}{4} w (a - a^3)$

Diagram gives reactions at Supports R_1, R_2 & R_3 for a Load "w" at distance "a" from Support R_1. Lower horizontal line represents different values of "a". Where vertical from given value intersects the different curves will give directly % of "w" which goes to each of the Supports.

For check $R_1 + R_2 + R_3 = 100\%$

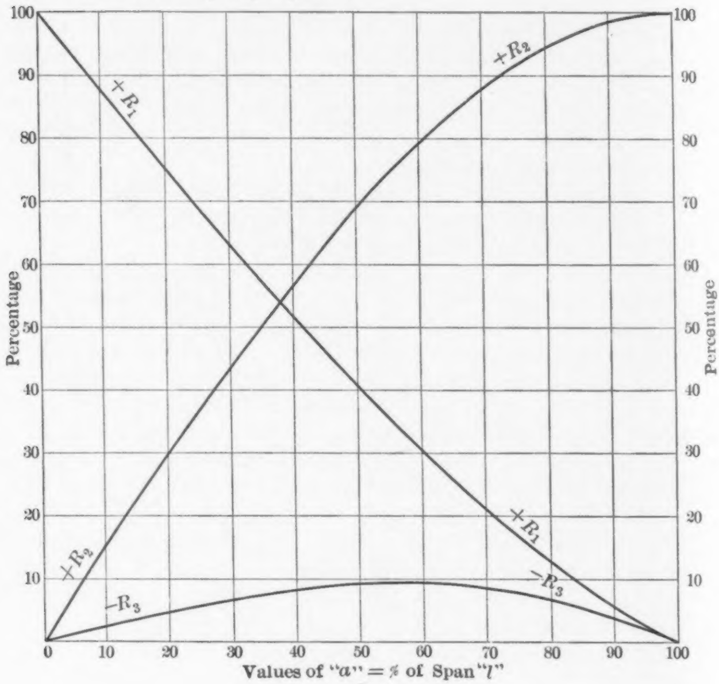


FIG. 4.

multiplied by the arms gives three equations. These, together with the equation, $R_1 + R_2 + R_3 = W$, make it possible to solve for the values of the reaction, as follows: Mr. Parker.

$$R_1 = + wb - \tfrac{1}{4} w (a—a^3)$$
$$R_2 = + wb + \tfrac{1}{2} w (a—a^3)$$
$$R_3 = - \tfrac{1}{4} w (a—a^3)$$

These three equations were used in laying out the curves in Fig. 4.

The results from the use of the curves are readily checked, for, having found the reactions for a given W, $R_1 + R_2 + R_3 = W$.

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PAPERS AND DISCUSSIONS

THE STRENGTH OF COLUMNS.*

Discussion.*

By Edward Godfrey, M. Am. Soc. C. E.

Edward Godfrey, M. Am. Soc. C. E. (by letter).—This paper is timely and important, not so much because it adds complexity to the already overburdened subject of the theoretical strength of columns, but because it affords an opportunity for a skeptical examination of the basis of all these column formulas, and particularly because any adverse criticism of the basic formulas will no doubt find a worthy champion in the able author. Mr. Godfrey.

In the technical press, the writer has repeatedly assailed both the Gordon-Rankine and the Euler formulas for columns. A paper on this subject, which if not denied ought to have revolutionized the subject, he had difficulty in finding a publisher to accept. It was not controverted when published, and it has not revolutionized the subject; the writer expected nothing of the sort; "what ought to be" and "what is" are separate and distinct things. It takes many years to pry accepted standards loose from a body of professional men, even though these standards are clearly proven false. In the meantime the writer has observed and demonstrated in his practice and reading that confidence in the Euler and Gordon-Rankine formulas has resulted in failure, as the error is so great.

When the writer was a student he swallowed the arguments of his textbooks largely because of the authority behind them. Since he has "put away childish things" he appreciates the fact that the highest authorities may err, and that error may be in the very subject that they know best. He accepted the apparent logic of the derivation of these formulas in those days, just as he would now probably accept what the textbooks state regarding the supposed strength of presumably

*This discussion (of the paper by W. E. Lilly, Esq., published in August, 1912, *Proceedings*, but not presented at any meeting), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Godfrey. reinforced concrete columns, which are constantly proving, by failing under a fraction of that strength, that such textbooks are wrong. Now, in reviewing his textbooks, he fails to discover any logical basis for either the Euler or the Gordon-Rankine formulas. If there is such basis, it is not stated in these books.

Textbooks say of the Euler load that "under this load the column just begins to deflect, and will under a constant load retain any deflection which may be given to it, within the elastic limit of the material." The writer can find no logical proof of this in the derivation of the Euler formula in these same books. It happens that the Euler load is one that will double any initial bow in a column. If an end load adds 100% to the initial bow of a round-ended column (more properly, one with knife-edge bearings), it will, by reason of this added bow, add another like amount because of that added deflection, and again another and another, and so on *ad infinitum,* or until the column fails. So that, whatever initial bow the column has to start with, it will fail at the Euler load. A column may be perfectly straight, that is, have an initial bow of infinitesimal amount: it will fail at the Euler load, and will not carry any more load than at its first measurable deflection. A similar column may have an initial bow of, say, $\frac{1}{8}$ in.; it will continue to deflect, but will not fail until the load is twice that which gives a total deflection of $\frac{1}{4}$ in. Both columns will sustain the same ultimate load (assuming that they are slender columns), though the originally imperfect one will deflect much more before reaching its ultimate capacity. These facts can be proven by the theory of flexure. They show one of the anomalies of the theory of columns. The writer believes that one would have to search through engineering literature a long time before he would find any statement of these facts, and yet they have a tremendously important bearing on the subject of the strength of columns.

The Euler load is independent of the tensile or compressive strength of the steel, depending only on the modulus of elasticity, which is practically the same for all grades of steel. The Euler load is the absolute maximum that any column can take, no matter how high the elastic limit or the ultimate strength of the steel may be, and two slender columns, of the hardest and of the softest steel, respectively, will have practically the same ultimate strength. These are also anomalies, and are very difficult to find in engineering literature. They also have an important bearing on the subject of the strength of columns.

While the Euler load is the greatest that any column could take, it has practical application only to slender columns. Short columns, by reason of the limiting compressive strength of the metal, cannot sustain loads approaching the Euler load, but will fail by crushing or

Mr. Godfrey.

buckling. Hence some other formula must be used for shorter columns. None can be correct, however, that shows greater ultimate strength for any column than that shown by the Euler formula, and herein is where the Gordon-Rankine formula is in error, at least, in its application in American books. Rankine does not point out this limitation in his derivation of the formula.* His statement: "The greatest deflection [of a rectangular column] consistent with safety is directly as the square of the length, and inversely as the thickness," is not sufficiently full. The deflection which counts is not the initial bow, which might be conceded to be constant for similar columns, but the resulting bow after the load is applied and equilibrium is established. There is no relation between this deflection and the dimensions of the column, for it is a function of the load itself. Any treatment that fails to recognize this is incomplete and is likely to result in error.

Dr. Lilly, in effect, ties up his Gordon-Rankine formula with the Euler formula when he recognizes that the deflection or curvature in the column will limit its carrying capacity. His values of p cannot exceed the Euler unit stress. The constant of his Gordon-Rankine formula is thus deduced from purely theoretical reasoning. This is eminently better than empirical determination of the constant as the latter has worked out.

A common value for the constant of the Gordon-Rankine formula for round-ended columns is $\frac{1}{18\,000}$. This, with a value of 50 000 for f, gives, for the ultimate strength of a column having a ratio of slenderness of 240, a unit stress of 11 910 lb. per sq. in. (Handbooks work this out for the busy user.) The Euler load for this column is only 5 140 lb. per sq. in. This is the absolute maximum load that any column could take, and yet a formula in general use appears to show that it can take 132% more than this. Here is the count which the writer would urge against the Gordon-Rankine formula, and he has known failure to result from confidence in this same formula with the constants commonly used.

The writer believes that the Gordon-Rankine formula fails to meet the needs of the practical design of columns, and that a straight-line formula is far superior. It gives results closer to those obtained from experiments, and there are several reasons why it should.

Columns, as commercially manufactured, are imperfect, of necessity, and a formula for their design should take into account this fact. They are not in true alignment, and their end connections are not always central. The writer has shown† that, if proportionate

* "Applied Mechanics," p. 361.

† *Railway Age Gazette*, July 2d, 1909.

Mr. Godfrey. imperfections are assumed in columns, a purely theoretical formula can be deduced, which, though very complex, gives a locus which is almost straight for a large part of its length and agrees closely with the commonly used straight-line column formulas.

The ultimate strengths of test columns fall away rapidly after the range of very short columns is passed. This is probably because of local crimping or buckling of the metal, but it is a fact which must be dealt with in the treatment of columns. The Gordon-Rankine curve does not take this shape, hence it fails to meet this condition. The straight-line formula does meet this condition, for the locus falls away from the start.

The straight-line formula has the further advantage that it discourages the use of slender columns. Slender compression members may be weak by reason of their own weight, or, if in a vertical position, an accidental blow may cause them to fail.

In the writer's opinion the whole subject of columns in engineering textbooks should be re-written, and its theoretical treatment simplified, instead of being rendered more complex. A large part of the engineering literature on this subject could be expunged with resulting benefit.

It is manifestly impossible to evolve a formula which will show close agreement with any comprehensive series of tests, for the reason that similar columns show discordant results. The exact strength of structural steel columns cannot be predicted, because imperfections of manufacture enter so largely in the results. Approximate results are all that can be expected, and simple theory answers this purpose just as well as the most complex theory ever devised.

In this re-casting of column literature, the importance of the Euler load should be emphasized, not as a load which the column can hold in equilibrium, conveying the idea that there is surplus strength in the column, but as the extreme limit of its carrying capacity.

Another fact of great importance which should be emphasized is that slender columns of all grades of steel are of practically equal strength. Working formulas should recognize this, and values should converge for long columns in low and high steels. Nickel steel struts of light dimensions are not economical, because their strength is practically the same as for soft steel, though they cost much more.

The converging of the strengths of columns of different grades of steel as the lengths increase has been illustrated by some tests* made by J. A. L. Waddell, M. Am. Soc. C. E. With similar columns of carbon steel and nickel steel in which $\frac{l}{r}$ was 27, the average strength

* *Transactions*, Am. Soc. C. E., Vol. LXIII, p. 230.

of the latter was 75% greater than that of the former; with others, in which $\frac{l}{r}$ was 81, the nickel steel columns averaged only 47% stronger. This indicates clearly the convergence to equality that theory proves must exist in slender columns of high and low steel. Mr. Godfrey.

On the Continent of Europe the Euler formula seems to be the standard for the design of columns. This is a grave error which American engineers do not commit. The Euler formula has no application whatever to columns of ordinary lengths, as used in bridges and buildings, for the values increase as short lengths are approached, and it would require steel of almost unlimited strength to satisfy the formula and hold up under the compression.

A short time ago, a gas-holder post, in a structure in Germany, failed, with disastrous results. The column was designed by the Euler formula, which was one of the errors made by the designers, for it was not (as considered by them) a slender column. The gravest error made in the design was the use of batten-plates instead of lattice. Another woful lack in the theoretical treatment of columns is that of emphasis on the extreme importance of some means of carrying

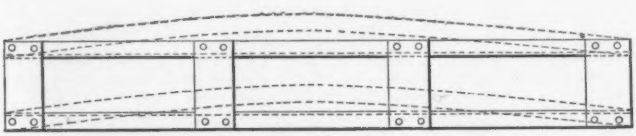

Fig. 5.

shear in both rectangular directions in the column. Batten-plates will not do this in any adequate degree. This column was made up of two 5-in. channels held together by a few pairs of small tie-plates. As the writer has pointed out,* these tie-plates, or batten-plates, cannot prevent the channels from bowing and acting practically as separate slender columns. Fig. 5 shows this column and how and why it could fail as a slender column. It is surprising that a high European authority, instead of condemning this flimsy construction, on the basis of common sense and theory, delivered the following:

"The use of tie-plated columns, when the section is assumed to be integral, may lead to constructions which do not afford adequate security under loading of unusual character."

Much is said of the impracticability of securing true hinged or pin ends on columns in testing them, and the idea is prevalent that most compression members are in fact practically fixed-ended in structures. This is more of the misinformation of engineering literature. It is much easier to get a practically pin-ended member in a

* *Engineering News*, July 27th, and September 28th, 1911.

Mr. Godfrey. structure than in a testing machine. In the latter, the rigid heads and friction on the pins hold the member under test almost rigid at the ends. In a bridge, the compression members are only insecurely held by other members as weak as, or weaker than, themselves.

A case which came under the writer's notice is that of the strut of a jib crane shown in Fig. 6. The designer considered this as fixed-ended, and proportioned it by the Gordon-Rankine formula. It is no wonder that failure occurred. The writer considered the strut as of the slenderness of one of the channels, and pin-ended. This is the only reasonable way to treat it. It could fail by the bowing of the two channels, as indicated. The single pair of batten-plates could offer but little resistance. The gusset-plates, to which the ends are attached, are more nearly like ideal pin-ended connections than pins would be, for there is practically no resistance against rotation.

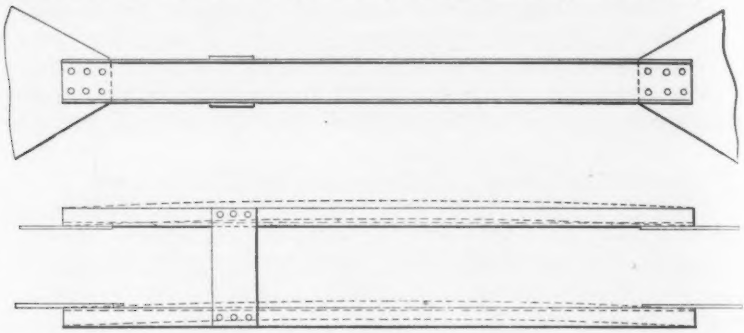

FIG. 6.

These are some of the things that could be written into the subject of columns to replace a vast amount of meaningless mathematics.

The subject of reinforced concrete columns has sprung up with a rank growth of mathematical nonsense which every great reinforced concrete disaster disproves. Tests are interpreted as applying to construction, while they do not embody the essential features necessary to safe construction. Columns utterly lacking in toughness are tested with infinite care (in testing) in order to preserve their evanescent strength; then such columns are built into a structure where toughness is an essential characteristic. Is it any wonder that the Engineering Profession is degraded by periodic wrecks, when its leaders show such lack of common sense?

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

STREET SPRINKLING IN ST. PAUL, MINN.
Discussion.*

By S. Whinery, M. Am. Soc. C. E.

S. Whinery, M. Am. Soc. C. E. (by letter).—This paper is so interesting and valuable that the reader reaches its end hungry for more information. Mr. Whinery.

It is so seldom that this branch of municipal work receives, from city engineers and city officials, anything like the attention it deserves or that is given to other departments of city work of no greater importance, that any one who takes an interest in the matter must welcome this account of an intelligent and efficient organization for street sprinkling in St. Paul. It is sincerely to be hoped that the author will favor the Society with another paper dealing with experiences and results. It would be very interesting to know about the practical working of the system, the efficiency attained, as compared with the usual unsatisfactory organization—or lack of organization—and the degree in which the service is successful in abating the dust nuisance and in meeting the reasonable demands of the public.

Particularly would engineers be glad to know the detailed cost of the service, reduced to units readily comparable with results in other cities.

The writer would suggest, as the most simple and satisfactory unit of quantity, 1 000 sq. yd. of street sprinkled once, and that all other elements of cost and service be based on this unit. The units commonly used (where statistics are reported at all) are often so indefinite or general as to be of little use for comparison. Thus, the number of miles of street sprinkled through the season is of little

* This discussion (of the paper by C. L. Annan, M. Am. Soc. C. E., published in *Proceedings* for May, 1912, and presented at the meeting of September 18th, 1912), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Whinery.

value unless one knows the widths of the streets and the number of times they are sprinkled daily, or rather the number of times they are sprinkled during the season.

The method described for assessing the cost of the service on abutting property owners seems to indicate that the width of the street is not taken into consideration. It seems proper that this should be done. Those owning property on a wide street certainly receive more service than those on a narrow street; and as property on a wide street is usually more valuable than that on a narrow street, the owners might justly be required to pay in the ratio of the work done, that is, the area of the street. To provide for this, of course, would introduce another factor into the computation of the assessments, but if the widths (or half widths) of the streets are known, the actual work of computation would not be increased greatly, though the unit on which the assessment is based would be changed from front feet to square feet, or square yards.

It is not stated how street intersections are dealt with in assessing the cost, though the inference that the cost of sprinkling intersections is taxed on the property owners seems to be warranted; nor is it stated how corner lots, sprinkled on two sides, are assessed.

As it is stated that street oiling is used to some extent, it would be interesting to know the relative cost, efficiency, and general merits of oiling and sprinkling in St. Paul.

It is not only in St. Paul that stand-pipes for supplying sprinkling wagons are regarded as nuisances. Their unsightliness might be overcome in most locations by using a valve and connection placed under the edge of the sidewalk and covered by a hinged plate. The chief source of dissatisfaction is usually the "sloppiness" around these stand-pipes. This is chargeable largely to the carelessness of drivers in allowing the tanks to overflow, and, where the hose is permanently connected to the tank, in allowing the water contained in it to waste on the street after it is disconnected. In this form of connection the trouble would be largely overcome if a valve were placed at the rear end of the hose, and closed before the hose is disconnected from the hydrant. Certainly this trouble can be overcome by the use of appropriate devices and reasonable care on the part of the driver.

While in most cities the municipality supplies, free of charge, the water used for sprinkling, and its value is not charged to the account, it is very desirable that the approximate quantity and cost should be reported, in order that a complete statement of cost per unit area may be deduced.

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PAPERS AND DISCUSSIONS

ENGINEERING EDUCATION IN ITS RELATION TO TRAINING FOR ENGINEERING WORK.

Discussion.*

By Messrs. Alexis Saurbrey, J. X. Cohen, George F. Swain, William J. Boucher, Almon H. Fuller, Walter Hinds Allen, C. H. Stengel, Charles Warren Hunt, Arthur H. Blanchard, Philip W. Henry, John C. L. Rogge, Charles H. Higgins, Charles B. Buerger, and Ernest McCullough.†

Alexis Saurbrey, Assoc. M. Am. Soc. C. E. (by letter).—It is very important to distinguish between "Engineering Education" and "Engineering Training." As to the first, education is, or should be, the common property of all civilized men, and the engineering school should not waste its time on the hopeless task of instilling true education, where home, environment, associates, and natural disposition have failed. Schools, colleges, and universities are struggling in vain when they attempt to "teach" taste, good manners, and gentlemanly behavior, if these qualities are not planted in the average boy at home, or, in many cases, acquired by less happy boys through natural disposition. "The well-read man is generally able to pose as a 'cultured' man," Mr. McCullough will have us believe. The writer denies this proposition, as well as the desirability of teaching young engineers to "pose." Certainly, it is a pleasure to meet a cultured, well-balanced, considerate man, and we cannot have too many engineers of that kind; but if an engineer is not so well-equipped, let him by all means avoid the deceit and shame of "posing." Mr. Saurbrey.

The writer, therefore, thinks that the sole problem of the college is to train. It cannot hope to train for the exceptional position at

* Continued from August, 1912, *Proceedings*.

† Author's closure.

Mr. Saurbrey.

the top of the Profession, but it can, and should, train for usefulness in the common, average case. The young engineer leaving college should be able to do correctly what Mr. McCullough properly refers to as the clerical work of engineering: Compute quantities, calculate stresses and strains, use the level, tape, and transit, and so forth. When, as a matter of fact, he cannot do that, the colleges are not solely to blame, for the preparatory school should have taught precision in algebra and arithmetic, which it does not do. Any really efficient reform movement in engineering education must begin with the home, and must fully consider the public school. With this attended to, the college will automatically adjust itself, and produce better engineers from the better raw material.

Nevertheless, some of the criticisms of the colleges are justified. No doubt the very broad training leads to neglect of details, and to superficial study. The remedy lies in an extension of the time for the purely engineering training, and in a curtailment of the volume taught, especially a reduction in the introductory studies of the first two years, whereby more time might be gained for the real engineering subjects. Such items as chemistry, physics, descriptive geometry, geology, and higher mathematics might, profitably, be reduced in volume, with the proviso that the subjects taught be really and thoroughly assimilated by the student, especially the simpler problems in analytic geometry and calculus.

The course in civil engineering, properly speaking, should certainly not be less than 2½ years (better 3 years), after the completion of the introductory studies referred to. All this time should be devoted to a most thorough drilling in fundamentals, with very little attention to generalities. The use of mathematics should be reduced to an absolute minimum, all complications being carefully avoided; understanding should be the goal aimed at, that is, intelligent application of thoroughly understood principles. Only a very few branches of civil engineering are on a truly scientific basis, and this fact might be taken advantage of, and engineering taught rather as an empirical profession than as a science; in other words, do not bother too much with the mathematical proofs of propositions which are, in reality, proved only by experience and experiment. The impossibility of transmitting telegrams across the Atlantic, the impossibility of flying, have been proved time and again mathematically, and yet the possibility was proved the next day in practice.

Without doubt, many teachers are trying to do just what is suggested here, and, if so, the writer feels that they are on the right track, and wishes that they would go still further. Many colleges, also, during the last few years, have given additional attention to the commer-

cial side of the question, and correctly so. While the writer certainly would be the last to excuse rank commercialism in anybody, he recognizes the fact that the engineer's principal purpose as an engineer is that of increasing values with as little expenditure as possible. The engineer is a wheel in a great commercial machine; as soon as he emerges from the modest initial incubator stage, he deals almost exclusively with business men; and the one question he has to answer is "what does it cost?" If, in addition, he cannot show that he himself is a fairly good investment, he will assuredly lose his job to the one who can. As it is, it takes a good while for the young engineer to satisfy himself and others that he is really worth his salary, and that is not right. It will be different when the graduate has been taught the immediately useful facts and formulas, and when he has ability to discriminate between extravagant and economical design of simple structures. Mr. Saurbrey.

It is not necessary to state that the college should teach its students the rudiments of bookkeeping and cost keeping. Instead, it seems that scientific management has been taken up. If hereby is meant "motion study" and such matters, incalculable damage will be done, for men are not machines, and should not be treated as such. Moreover, the writer believes that this fad will be a thing of the past in a few years, and the college should be very conservative in introducing such matters.

Mr. McCullough's paper, as well as his recent book "Engineering as a Vocation," are most valuable and interesting. They disclose in a clear, concise, and wholly unprejudiced manner the very foundation for that dissatisfaction so common among recent graduates, and so often expressed by them in the engineering press. It is not only a question of pay, for engineers are as well paid as attorneys and doctors, and much better than teachers or ministers, all of whom have to put as much time on their training. It is mainly a question of competency, of ability to render service in the world as it is—the engineer seeing the great opportunity he has for service while the public does not; but the public will. The engineer of to-day is a pioneer who must clear the forest of misunderstanding, indifference, and inertia, and that takes time. In addition, the fields opened by the modern testing machine, indeed, by the modern spirit of research, have not been properly explored, and we still suffer from many "ifs" and "buts" to be solved in the future. The problem of writing good textbooks is no easy one, when new research makes old truth obsolete over night, and, as long as the teacher must study the changes in the fundamental theory, he is greatly handicapped as a teacher. For this very reason, reading knowledge of foreign languages is almost indispensable to an engineer who wishes to be up to date in his specialty; but

Mr. Saurbrey. they should be taught in the preparatory school, and along practical lines, not in the college.

On the surface, the problem raised by Mr. McCullough seems possible of satisfactory solution; but in reality it is one closely connected with the home and the public school, and, therefore, with the community at large. The battle-cry of to-day is reform, the enthusiasm behind the guns is dissatisfaction. One question, indeed, suggests itself strongly: Is not the failure of the weak, and the survival of the fittest, a principle against which we are battling in vain? one that will exist even if the most ideal vocational training were given? Surely those who are now dissatisfied engineers would otherwise be dissatisfied mechanics, and no happier than they are at present.

Mr. Cohen. J. X. Cohen, Jun. Am. Soc. C. E. (by letter).—The author aims in the proper direction. He seeks to serve the student first and then his future employer. The sound, fundamental, non-specialized technical course which the author recommends makes the student broad and receptive, rather than narrow and exclusive.

It is encouraging to note that the course outlined emphasizes so greatly the study and the value of English. By English is not meant the polished literary language of the library, but the sturdy style of the council chamber and the business office. To the great detriment of the engineer, his English course has usually been made a minor one, and very often neglected at that. That is a very serious situation, and calls for rapid remedial measures. Certainly, engineers should first know how to handle materials, but what more valuable materials are there than men, and what means of communication between men exists, other than language? Even when engineers deal with each other directly, what matters it how well their minds may operate if the thoughts cannot be transferred clearly and correctly? We all know men who have good ideas and excellent thoughts which are hardly ever realized, solely because they are not plainly stated. The ultimate significance of the idea cannot be quickly made clear to others, and it dies before it develops.

The author considers that course in engineering most beneficial which permits of alternation between class-room and field, between school and shop. The writer, having received such a training, and having further observed the comparative effects of the older method of training, heartily endorses the newer.

There are several technical high schools in New York City, the graduates of which are equipped for entering either the engineering school for advanced studies or the engineering office for practical work. It may be of interest to state that a very large percentage of these graduates goes immediately into actual work rather than into

college, without, however, having abandoned the idea of a higher technical education. Mr. Cohen.

Having secured a position which their technical high-school training qualifies them to hold, they next enroll in the evening engineering course of the Cooper Union for the Advancement of Science and Art, or some similar institution, of which there are also several in New York City. Here they spend their evenings for a good many years—five years at Cooper Union—in hard, arduous, and comprehensive study, supplementing the practice followed during the day with the knowledge gained at night.

This method of study makes for the greatest good. The co-ordination of class and field produces results which are harmonious and well-balanced. Studies are pursued with the greatest interest; their immediate application in practice is either actual or plainly discernible, and their utility needs no emphasis by the instructor. Very often the problems arising during the day may be worked out in the laboratory or class-room during the evening. This produces impressions which are vivid and knowledge which is secured. At times the pace in the class-room would appear to the regular day school instructor to be extraordinary. This combination method makes speedy and successful studying possible.

Such a combination course helps a man financially in several ways. For one thing, he is self-supporting throughout all the period of study, despite the fact that such a course may take a longer time than the so-called regular one. He is employed constantly, and not only during school vacations. This surmounting of the financial barrier is valuable to the Profession, for otherwise many good men would find it hard to prepare properly for practice. For another thing, the combined day worker and evening student finds that as his technical knowledge increases his employers correspondingly increase his compensation. As he observes his increasing pay, he notes the effect of his spare-time study on it, and, as a result, the incentive for further and more concentrated study is greatly strengthened. Better than a good report card is a larger pay check, for while the first predestines the other as an eventuality, the second is the actuality. Not all men, especially in engineering, work for gain, but the stimulating and encouraging influence of tangible recognition is highly beneficial. Finally, a man is helped financially—as well as in numerous other ways—by being kept so busy that he finds no time to get into mischief.

The graduate of the combination course, when he receives his degree, is handed a certificate which shows that he has demonstrated his capacity for hard, continuous, single-centered work. If he had not possessed this ability at the beginning of the course, he would never have reached its end, except through the inculcation of that faculty

Mr. Cohen. in him by the example of his fellow-students. If for nothing else, such a course is of value as a demonstration of the true capability of the man to do diligent work and his real capacity for conscientious, continual toil. Too few men realize until very late in life the enormous amount of work that can be accomplished without undue fatigue by strict adherence to a carefully planned programme. Further, the utilization of spare time for self-improvement is taught in an unforgettable manner, and as the graduate must necessarily be a student after graduation, by pursuing the combination course he learns how and when and what to study after his college days are over. The waste of spare time prevalent among many young engineers is great, and it is a waste which is a direct result of the lack of early training in spare-time study. The student of Cooper Union learns to work even when traveling on trains, unconsciously following the example of the most eminent consulting engineers in active practice. He who learns how to utilize all his available time efficiently has a splendid start in the race toward professional success, which ordinarily can only be attained by continual concentrated application; and to this type of application the graduate of the combination course is no longer a stranger.

The student who is engaged simultaneously in the study of engineering and its practice enjoys a great privilege. He can ascertain whether he has that aptitude and inclination for engineering, which, to a great extent, is vital to success long before he has invested much money in his course or much of the even more valuable time in its study. He has the advantage of being able to decide whether engineering appeals to him as a life work at a much earlier stage than the regular school student. The number of students who are graduated from the regular course, and fitted by training for engineers, is now very large, but of these only a fair percentage is fitted for it by natural talent, inclination, and equipment. Many realize this some years after graduation, but then it is too late, from their viewpoint. Having spent so many years in preparation, they fear to see all their efforts go to apparent waste. They also greatly fear the possible ridicule of their friends at their early recognition of and submission to failure in their chosen calling. Such motives as these keep many men in the ranks until, by force of circumstances, they are forced out or forced up. For a long time, however, they encumber the lower rungs of the ladder, making it harder and harder for themselves as their numbers grow, and also more difficult for the young engineer of future merit to obtain a foothold; but whether or not they stay in the Profession, they have suffered a grave economic loss. In this loss the community at large is also a participant, and it is to relieve

the public and the prospective engineering student from as large a measure as is possible of this partly preventable loss that the combination course is advocated by the writer. Mr. Cohen.

The Profession is benefited directly by the combination course. Few but the strong, the steady, and the persistent complete such a course, so that the process of weeding out starts at once and has just that much longer to operate. It is an effective block to the lazy, unambitious young man, who would stand but a slight chance were he to enter active practice. If time were available, the writer would like to discuss the role of the engineering teacher in the school attended by students who are at the same time in active practice, but suffice it to say that these teachers must be mentally alert, on the very *qui vive* for the latest and best information and methods of its presentation, and altogether on a high plane, in order to maintain the necessary leadership over their students. Otherwise, they will find themselves being taught by their own men, who, in some details, may be better acquainted with the subject. To the Profession, the value of such a high teaching tone need hardly be pointed out. Furthermore, the student working at some branch of engineering, as he nears the end of his course, can decide for himself whether he prefers that particular branch as his future specialty. He can then begin to supplement his training in the engineering fundamentals by a course of study in his chosen specialty. Such an early decision as to the choice of a life work, if made carefully and discriminately, makes available more time for the attainment of that greater knowledge and understanding of a subject which produces the real specialist. Finally, it starts the student under auspices which will operate for his individual betterment and for the benefit of the Profession.

It may be urged that the grind of the combination course leaves the student no time to attend social functions. In a measure this is true, and hence beneficial, as previously pointed out, but it is not altogether true. The writer's experience and observations lead him to believe that all necessary social functions can be attended without hampering seriously the work at office or school. The course is not one grueling grind, for it is interspersed with a number of holidays and a long summer vacation. By careful and far-sighted planning, a time for almost everything that is reasonable can be found. Of course, numerous social activities, which make up a part of the college life and take up an appreciable part of the student's time, are necessarily curtailed or completely eliminated. The advantages to the student of such comparative freedom from the disturbing and, at times, harassing influences of many social engagements need hardly be pointed out. The impression, however, should not be gathered that the combination-course student is a "grind" simply because he

Mr. Cohen. lives the concentrated life demanded in large part by modern industrial conditions. His lot is not a hard one, and, being always busy, he is in general always happy.

Mr. Swain. GEORGE F. SWAIN, M. AM. SOC. C. E.—The speaker is always very glad to read a paper on education by a practicing engineer, and always derives some good from it. This is true of Mr. McCullough's paper, but, at the same time, there are certain points in it with which he does not agree.

Mr. McCullough states that we must distinguish between engineers and engineering teachers. As Professor Constant has pointed out, the majority of engineering teachers at the present time are or have been engineers. Many of them are practicing and teaching at the same time; and, as Professor Constant states, the younger men who take up teaching are drawn generally from the ranks of practicing engineers. These teachers know probably better than any one else how a curriculum should be drawn up, because they know, not only what the practicing engineer wants, but also what it is practicable for the school to do. It is impossible for a man who has not tried to teach to draw up a curriculum which will work well; he almost always forgets that the problem of engineering education, or of education in general, is not an engineering problem, but a human problem. We talk about the teaching of engineering, but we probably forget what we were, or what the ordinary boy is, at eighteen or nineteen, and we cannot very well theorize unless those things are kept in mind.

One of the most important things to remember is this: Mr. McCullough speaks about the engineer drawing up a specification of what he wants in a man, and the schools filling that specification. The speaker does not think that an engineer can draw up a specification of what he wants, and if he can, the schools cannot fill it, or at least they cannot guarantee to fill it, because they can only teach the student what he can do for himself. The teacher does not give the student knowledge, he shows him how to get it; and if the student does not want to accomplish anything himself, the teacher cannot force him to do it.

The manufacturer, who, for instance, wants to make a spoke of a wheel, can take a piece of wood and fashion it into the proper shape. Now, it may be said that the teacher's raw material is the student, and though the teacher knows what he wants to make of him, he cannot control his raw material; he cannot cut away here and add there, he can simply show the student what he can do for himself. The most important thing in teaching, therefore, is not what shall be taught, but how it shall be taught. That is a truism, a platitude, but it is what we must keep in mind. The important thing is to have the proper atmosphere in the school, in order to make the young men

realize that they have great opportunities before them, and that they are being offered a chance to gain physical, mental, and moral qualities which will fit them to meet the problems of life. Mr. Swain.

When the employer of engineers asks for an assistant, he does not care very much what the young man knows; that is of the least importance. He wants a man who is faithful, who is of good character, conscientious, who can think straight, who will not be anxious to stop work as soon as the bell rings, who will be loyal to his employer, who has "gumption," and who can meet emergencies. The amount of knowledge he wants in the young man at the start could be given to him in a very short time. It is the other qualities which are important. The school, therefore, should pay particular attention to the cultivation of the proper atmosphere.

The speaker, of course, has his ideas in regard to what engineering schools should be, and they are very simple. The trouble with the schools is that they try to carry their technical instruction too far; they are narrow; they do not realize that the young man, in starting his career, will not need much knowledge, and if he has the little that is needed, and the other qualities which have been mentioned—the ability to think straight and to take up a new subject and master it—he will be ready for his job, and for promotion, whenever the chance comes. The majority of schools, therefore, should pay more attention to fundamental principles, and not try to carry details quite so far in particular branches. There ought to be a few schools for post-graduate instruction for men who are qualified and can take the time for a more thorough education; and with such an arrangement and the proper kind of instruction, engineering schools should be able to turn out men who will be satisfactory to employers.

The engineering schools are turning out good men to-day, but, like everything else, they can be improved. The schools realize this, and each is trying to remedy its defects as far as possible. One trouble is that parents do not co-operate sufficiently with the schools, the prevailing tendency being to throw everything on the latter. Parents should earnestly co-operate with the school in making the students realize the great opportunities offered them, and the fact that they must work hard; this does not mean to work all the time, but to work hard and endeavor to utilize their time to the best advantage.

Mr. McCullough and one of those who discuss his paper refer to the fact that there are numerous instances in which a man finds himself in after life practicing a different branch of his profession from the one he studied in college, the inference seeming to be that this is a very bad thing. The speaker has never been able to consider it so. The main thing is to follow a line of study in college which will give a man the qualities which he needs to enable him to meet the problems of life. The speaker has had engineering

Mr. Swain. students who subsequently became ministers; others who became lawyers; some who became artists; one or two who have become economists; and others who have gone into business. He has talked to many of these men, and has yet to meet one who has regretted his engineering education. They all admit that such a training gave them what was more valuable than anything else, namely, the ability to concentrate, to work hard, and to get results.

In fact, the speaker has almost come to feel that the study of engineering is about the best training for a young man, no matter what his future career is to be; and if he had a son, whether he was going into business, into the law, or into anything else, he would select such a training for him, because he thinks it would give him, better than any other, those powers which he would desire him to acquire. Besides, he would be dealing with every-day things. Engineering is practical, and engineers are dealing continually with electricity and with mechanics. If these views are correct, we should not be surprised to find many men taking courses in civil engineering and afterward practicing as mechanical or electrical engineers, or *vice versa*. There are very few men who, when they enter college, can feel sure that they are fitted for any specific branch of the Profession. They may know that they like engineering, but their future career is very likely to be determined by some trivial accident. If a man has a good training to start with, and the character and the power that he ought to get at school, he will succeed, and he ought not to be the subject of criticism because he takes up some other branch of work.

With reference to the usefulness of modern languages to the engineer, Mr. Boucher and the author think that modern languages ought not to be required in engineering education. In regard to that the speaker disagrees with them entirely. Recently, he attended the Sixth Congress of the International Association for Testing Materials held in New York City. There were several hundred men at that Congress from all over the world, including the most prominent representatives of that branch of the Profession from almost every country of Europe, one from China, and one from Japan. Almost all those men could speak English; most of them could speak two modern languages. Mr. Henry M. Howe, one of the most distinguished of American engineers, the President of the Association, made his address of welcome in six languages, though the speaker does not suppose that he speaks each of these six languages fluently.

Now, if it is believed that the engineer should occupy a high position among men, not merely that he should be able to do his engineering work properly—building his bridge, laying out his road, or designing his power station—but that he should occupy a high position among men, it appears that a knowledge of such things as modern languages should be encouraged. It is, of course, perfectly true that a

man can become just as good an engineer, in a purely technical sense, without knowing anything of modern languages, of economics, or of a great many other things, but a very high standard for the Engineering Profession should be demanded and maintained, not simply in engineering, but among cultured men, and if that is done, a knowledge of at least one modern language, and preferably of two, should be encouraged. Therefore, a student who is graduated and takes a degree from an engineering school should have at least a good reading knowledge of one modern language. The man who cannot get that, can take a special course and get through technical instruction, but the colleges and professional men of to-day aim for something broader than mere technical training. Mr. Swain.

WILLIAM J. BOUCHER, ASSOC. M. AM. SOC. C. E.—This paper is both interesting and timely. Changes have occurred and are occurring in all lines of business, including engineering, and why should not corresponding changes take place in preparation for business and engineering practice. The speaker agrees with the author that schools and professors should aim to fit their graduates more closely for the work to be undertaken immediately after commencement. Very clearly does the speaker remember his first days in engineering work—at the very bottom—and the many very ordinary things he did not know. Mr. Boucher.

The author expresses the belief that engineering schools of the future will require a minimum of six years' work, of which two years will be spent in the preparatory school, but adding two years to the entire time required in preparation for the life work. The speaker believes that such a lengthening of the course would be a mistake. The average age of entering students has increased steadily, due to the increased entrance requirements, until it is now generally about 19 years, which, with a four years' course, makes the graduate 23 years of age; it does seem that this is old enough to start life's practical work, without requiring an additional two years, making him 25 years, or possibly 24 years, if he has been fortunate enough to finish the course in three years. Very few men would be able to do this, for a variety of reasons, chief of which would be the financial one, and those who had their tuition paid by parents or others would hardly feel the stimulus to do it in less than the prescribed time. The speaker was graduated at the age of 21 years from one of the best known mechanical engineering schools, after 14 years of continuous study, and felt and still feels that that was quite late enough to go out into the world. The following advertisement, copied from a recent issue of one of the leading engineering weeklies, appears to emphasize this latter point:

"Position wanted by graduate civil engineer, 25 years, one year's graduate study, open for permanent position in any line of profession,

Mr. Boucher.

locality immaterial, experience in reinforced concrete construction and sewer design."

Doubtless, that advertisement will be read by several prospective employers who would much prefer that the applicant should have three or six months' practical experience, rather than a year's graduate study.

Mention is made of the fact that colleges admit all who apply and can pass the entrance examinations. This is true, and, as a result, many young men enter engineering courses who are unfitted mentally and temperamentally for that line of work. It seems to be such a waste of good time and effort to instruct young men in technical lines when they would make better mechanics, carpenters, clerks, or farmers. Before applying for admission, a young man should be made familiar, by parents or teachers, with the qualities essential to success in engineering; he should be observed and questioned as to his liking for and ability to solve mathematical problems, and, by various tests, his qualifications should be known to those who would be in a position to advise him in regard to his life work; for, although the engineering and technical studies will not harm him, and in certain ways will prepare him for any work, it would surely be much better for those who do wish to follow engineering as a life work if the classes contained only those and were not overcrowded with many who belong more properly in academic courses and do not care for the engineering training or propose to follow that Profession. This leads very naturally to the observation that so many graduates of engineering courses are found in lines of work in no way related to their training, and it would be largely eliminated if advice and thought were given to the future of the graduate, rather than to the haphazard method, so frequently pursued by parents, of sending their sons to attend an engineering school, because it "seems to be the proper place," or "the proper thing to do."

In a recent address, Alexander C. Humphreys, M. Am. Soc. C. E., President of Stevens Institute, said:

"Many fathers and mothers come to me and tell me that their boys have a natural bent for engineering. Why? Well, they show great aptitude for making electric bell connections, or they are very skillful at the lathe. I generally tell them this: Will your boy apply himself to the hard study, perhaps, to him, the drudgery of mathematics and science? Otherwise, turn your attention to making your boy a good mechanic. The boy must have capacity for mental application besides manual dexterity."

In regard to lengthening the course beyond four years, Dr. Humphreys says in no uncertain language:

"If the course is to be lengthened, who shall determine its duration; if five, six or seven years are needed, then why not seventy, for

a genuine student can always learn. One of the disadvantages of a college training, which must be offset by the greater advantages, is that students get to relying too much on their college training." Mr. Boucher.

Further, technical schools are seldom endowed as liberally as the older and better-known universities, and it is a well-known fact that the cost of a student's education is more to the institution than the latter receives in tuition, consequently, the larger the classes the more the institution runs behind in operating expenses, and, for that reason, if for no other, as many students as possible should be deflected into those colleges giving cultural or academic courses. Another very good reason for keeping the classes small, is that, by so doing, the professors come into closer contact with their students, which is always a great advantage to the latter.

On page 647,* the author gives a list, more or less complete, containing his ideas of entrance requirements. This list contains almost the identical subjects required for entrance to Stevens Institute in 1892, in addition to geography (political and physical), United States history, rhetoric, composition, and—probably most important of all—arithmetic. This last, for some obscure reason, the author seems to have overlooked. To the speaker, however, it is a most important subject, one which is constantly used, and in which proficiency and accuracy are most essential, and its use should not be subordinated to the slide-rule or "guessing stick."

As for foreign languages, the speaker is in accord with the author; they should not be required during the course, in spite of the view of one very much respected professor, who held the opinion that the study of foreign languages gives relaxation after the hour of mathematics or physics. A reading knowledge of modern languages is certainly an advantage to the engineer. It should be acquired in the high school, however, and, in order to keep up the practice, reviews of certain foreign technical papers might be required sufficiently often to insure that the student was not losing what he already had. The difficulty in after life is that language studies, probably not any too thoroughly taught in college, are completed (so-called) one or two years before graduation, and, when the latter occurs, the graduate is so "rusty" in his languages that the reading, being anything but easy, is consequently neglected and soon dropped completely; for the busy engineer in practice has all he can do to read a portion—a very small portion—of American technical literature, which each week and month is demanding his attention.

The course in engineering should be made pre-eminently practical. Its use in the future should be kept constantly in view, and

*Proceedings, Am. Soc. C. E., May, 1912.

Mr. Boucher.

those subjects which will make the fresh graduate useful to his first employer should be elaborated—drafting and drafting-room methods should be insisted on and required. For five seasons the speaker was instructor in a New York City evening school, teaching mechanical drawing. He aimed to make the course useful and practical, devoting only a short time to mere drawing, but advancing the students rapidly to sketching from objects, then drawing the same in a neat and accurate manner, and finally tracing in ink; and, though a season's course lasted only six months, he has the satisfaction of knowing that several of the students, who had never before been in a drafting-room, obtained employment as tracers or junior draftsmen after their one season's course.

The author outlines a course of general engineering study covering four years and designed to produce graduates who shall be well educated on broad lines and acquainted with much that is actually required in their future work. The speaker finds very little to criticize in the work outlined. For several years, Stevens Institute has required, as a part of the course, attendance at lectures and recitations on "business practice," in which attention is given to accounting, depreciation, analysis of cost, specifications, estimates, contracts, and appraisals.

There is probably a diversity of opinion in regard to thesis work, but when properly conducted, and not consuming too much time, some good results may be achieved, for instance, in carrying out a test of a power station at a distance from the college, where the students must rely almost wholly on themselves.

In closing, the speaker desires to mention an incident, which occurred at almost the beginning of his practical experience. Application had been made to a rather prominent consulting and contracting engineer of 1897, who is still in practice, to enter his employ in a minor capacity. The answer, in letter form and preserved as a memento, reads as follows:

"There exists at present no vacancy in my office, but my experience with college graduates has been such that I do not care to repeat that experience."

Fortunately, this attitude is rare, and will become rarer as the products of our colleges and technical schools prove their worth by being immediately useful after graduation.

In conclusion, the speaker desires to make this criticism of all the discussion by professors—that they seem to overlook or ignore the ultimate object of all the teaching, namely, to enable the graduate to secure a position in engineering work promptly after graduation, for that is what 99% of the graduates need. Professors are much inclined to require a too highly finished product, rather than

a working knowledge of essentials. Engineers in practice know what they lacked when they started out in the world; they also know of the hours spent on work required in college, which has never been hinted at or needed in practice—work which can properly only come after years of experience in the active practice of the Profession and is only entrusted to those who have obtained standing and reputation by their years of experience; hence it does seem that engineers are very distinctly qualified to have a voice in the making of the curriculum which is planned for the education of their future assistants. Mr. Boucher.

ALMON H. FULLER, M. AM. SOC. C. E.—Mr. McCullough has stated that engineering teachers should get together and standardize the courses of instruction. That sounds well, but he seems to have overlooked the fact that each man will have to deal with the situation as he finds it in his respective college, especially in other departments, such as physics, mathematics, and chemistry; and even though they should agree on a standard, there would be difficulty in taking it home and applying it. It is possible that some progress could be made in that way, but the conditions which exist would cause considerable difficulty in effecting a uniform change. Mr. Fuller.

The author also suggests a sequence of the various subjects which differs entirely from that usually followed. By this he hopes to give a certain amount of practical work the first year in subjects which will permit the students to do certain work during the summer, with the thought that if a man stayed by it without coming back to school perhaps the entire Profession would be the gainer. There has been much discussion on the proper sequence of subjects in an engineering curriculum. The usual order is to give much of the so-called cultural work first. Perhaps many would agree that this should be distributed throughout each year.

In talking with some of his own students, the speaker has noticed a greater inclination to take general work in the latter part of the curriculum than in the first. If given in the first part, it is thrust upon them; if available later, many will take it willingly. The speaker has heard practicing engineers suggest such an arrangement. Whether or not this is the better plan seems to depend largely on the spirit that can be instilled in the students at various times.

Mr. Green has well said:

"Just what subjects are studied by the one being educated is a secondary matter; the chief concern is that the study shall be inspired and directed in such a way as to develop qualities which further happiness, efficiency, and capacity for social service."

When every instructor recognizes this, and realizes that it includes fundamental training for general resourcefulness—culture if you

Mr. Fuller. please—much progress will have been made. This is of greater importance than the particular arrangement proposed by the author.

Mr. McCullough suggests that a specification for engineering education be written by engineers. Professor Swain thinks that would not be practicable. Perhaps it would not be. The speaker can see many objections to it. However, as an engineering teacher, he would like to see the specification. He would welcome the opportunity of examining it, of comparing it with the present curricula, and of attempting to adapt it to the conditions that exist in the institution with which he is connected. If a representative committee of engineers would take the trouble to write such a specification they would deserve the thanks and possibly receive the approbation of the teachers. As Professor Swain has said, unless the men who write it were very closely in touch with the engineering colleges, it might not be very useful, but it seems to be entirely possible that it might bring out many points which engineering instructors could adopt with splendid advantage.

Office atmosphere may well be kept in mind in conducting courses in drawing and design. At the same time, it will not do to lose sight of the fact that, in the office, the intent is to mould the entire force into a smoothly working machine which will produce the greatest output; while, in college, the purpose is the development of the individual.

Mr. Allen. WALTER HINDS ALLEN, M. AM. SOC. C. E.—In the first part of the Nineteenth Century the young man who desired to become a lawyer secured his professional training by going into some law office where he would read law for several years. Later, law schools were founded, and, by attending one of these, a much better legal education was possible. These methods, however, did not afford a broad education, and, nowadays, the majority of law students first acquire a general college education, waiting to get their technical education until the age of twenty-two or later, when the mind of the young man is so much better able to comprehend and master the more intricate technical problems. Some of the modern law schools will admit only students who have received a Bachelor of Arts degree or the equivalent. These schools recognize the fact that general education is essential in order to produce the best lawyers and citizens; and that the man of twenty is not able, in most cases, to get the full benefit of his professional study.

This same condition is true to some extent in the study of medicine. Of course, there are and always must be schools of law and of medicine which admit students whose education has not advanced beyond the high school. Not all young men are able to afford the time or money necessary for a college course, and it would be most unjust to deprive them of an opportunity of entering these pro-

fessions. It is generally recognized, however, that such a course is a desirable preparation for professional study. Mr. Allen.

At present the engineering schools of the country are at that former stage of the law and medical schools, when a previous college education was not a requisite for admission. The Engineering Profession is behind its sister professions in this respect, for a good general education is just as essential a preparation for engineering study and to produce the best engineers as for any other profession. Such general education need not be exactly the same for all professions. For one who intends to study engineering, much preliminary scientific study may be undertaken in mathematics, physics, and chemistry; but history, economics, literature, modern languages, and rhetoric should receive considerable attention. These subjects will prove of value, not only to the engineer in practice, and particularly as he attains more prominence in his profession, but they add to his culture and ability to stand well among his fellow men. They increase his power of enjoying the higher things of life.

An undergraduate college course is completed ordinarily at the age of twenty-two, at which time the young student, having reached the more serious period of life, is ready to take up the technical preparation for his life work. If he has finished his studies in pure mathematics and other elementary subjects, he can get a thorough engineering education with two or three additional years of study.

In another respect, the engineer may well profit by the example of the lawyer or doctor. After graduation it is a very common thing for these men to enter law offices or hospitals and work for one or two years with little or no compensation. They do not so much consider the financial side as the opportunity afforded to observe the best practice and to supplement their study at the professional schools. In the speaker's opinion, it is entirely wrong to assume the attitude that the man who has just completed his technical school course should begin immediately to earn good pay. He is not yet of any great value in his profession; the man who has not had the opportunity for education, but has started his practice at an early age, is, for a number of years, of much greater value to his employer. The trained man, however, has far greater possibilities in him, and nine times out of ten becomes the better engineer after he has had some years of practical experience. He himself should realize this and be content in his first years to make monetary compensation a consideration secondary to securing the best experience.

The speaker had occasion last winter to investigate the opportunities offered for certain young men, technically trained, and graduates of an engineering school, who had had two years of practical experience, to take a course of study that would give them a broad civil engineering education. These men were about twenty-five years of

Mr. Allen. age, good students and well equipped in mathematics and some branches of civil, mechanical, and electrical engineering. As far as the investigation disclosed, there is only one Eastern university or engineering school which has a regularly organized graduate school of engineering. This has been started recently, and marks, in the speaker's opinion, an epoch in engineering education in the United States. The number of young men who take up the study of engineering in a graduate course, after obtaining a general college education, is steadily increasing, and the opening of this graduate school is an index of the trend of engineering education.

It is a good omen, too, that this and other engineering societies are taking interest in the education of those who later will become engineers. The members of the Profession by their advice and interest can exercise a strong influence in securing the best training for their successors. This cannot be done effectively by bringing pressure on the schools themselves and by trying to dictate what they shall teach. The schools will furnish that kind of education for which there is a strong demand from the students themselves.

Outside engineers can do far more good by using their influence with young men who are intending to become engineers, by inducing them to secure a good general education first, and to pursue their technical studies afterward. The practicing engineer should encourage the beginner to take a broad view of his profession, to look to the future, and to map out his early training and practice with a view not so much to immediate financial success as to attaining ultimately the top of his profession.

Mr. Stengel. C. H. Stengel, Assoc. M. Am. Soc. C. E.—In order to substantiate some of the facts brought out by Professor Swain, pertaining to the statement that engineers should be graduated at the age of twenty-one in preference to a more advanced age, to give them an early start in the Profession, the speaker would state that he has had in his service a number of young graduate engineers, and, after careful observation, has found that their intellects are at a more advanced stage of development, their work more accurate, and themselves better men on the average, at the ages of from twenty-three to twenty-five than at twenty-one. The more mature the mind of the student at the time he is laying the foundation of his career, the greater are his intellectual powers, principally in absorbing and retaining the knowledge he is gaining, to develop his logic and reasoning.

When the young man enters college intending to take up Engineering, his course should consist in mastering thoroughly and conscientiously the fundamental principles which form the basis of the Profession in all its branches; then, with his power of application, he should be able to fit himself for any of its branches, and his rise

will soon be assured, if his energies, resourcefulness, and ambition are applied to his work. Mr. Stengel.

As stated, it is the personality and self-reliance of a young man entering the engineering world, together with the thoroughness in which his mind is developed in not only the fundamental principles underlying his Profession, but in careful analysis and accuracy in the performance of any work he may pursue, that mean success; and to accomplish this he should have the full confidence of his tutors and the co-operation of his parents (as stated by Professor Swain) in the moulding of his career.

CHARLES WARREN HUNT, M. AM. SOC. C. E.—The general subject of the education of the engineer is of great interest to the speaker, inasmuch as, for more than twenty years, he has been in a position which has enabled him to form an opinion of the results of modern technical training. Mr. Hunt.

Professor Swain has stated certain logical, broad, and proper basic principles on which engineering education should be founded, nevertheless, in the speaker's opinion, the tendency of the modern technical school is to become more and more narrow.

A boy who wishes to become an engineer must decide, practically upon matriculation, which special branch of this great Profession he will follow: Civil, Mechanical, Electrical, or some other. During the course in whichever specialty he chooses, he is forced to spend many hours in working out details of that specialty (in many cases without even a suggestion of a study of modern languages, history, literature, or in fact of any of the humanities), and, after four years of hard grinding, is graduated as the particular type of engineer indicated by the title of the course pursued. He must then secure a position for which that preparation is supposed to have fitted him—he has no other option—and then follows a period of years during which, in the struggle for existence, his nose is kept so close to the grindstone that he has no time even to look about him for broadening influences; so that, when he reaches the age at which he should be most productive and efficient, he is not fitted to take and keep the position, in the social, political, or business life of the community in which he lives, to which his intellectual attainments and constructive skill entitle him.

It is trite, but true, to say that the engineer is the pioneer of all civilization, as well as one of the most important factors in its advancement; and it is then most natural to inquire why his position among his fellows is not commensurate with his achievements. In the speaker's opinion, it is because he is not enough of an all-around man; he is not broad, not capable of thinking clearly and quickly along any other lines

Mr. Hunt. than those to which he has given up all his formative years. He does not, therefore, succeed in impressing his personality on his fellow-man, although he has not the slightest difficulty in so doing on his fellow engineer.

The speaker believes that the modern system of engineering education is, speaking broadly, responsible for this condition. He does not know enough to attempt to discuss any of the details of curricula or class-room, but would like to go on record as believing that the specification for a properly equipped technical graduate should not be that he should be able immediately on leaving school to be valuable to an employer in any specialty, but that first of all he should be full to repletion with knowledge of the fundamental laws and principles of the exact sciences on which the sound practice of engineering in all its branches must be based; and, in addition to this, his attainments outside of technical matters should be broad enough and fundamental enough to enable him to become a man of the world. It is time enough for him to specialize when he has found out what he is best fitted for, and what his opportunities are. To be successful, an engineer must not only be able to do the technical work which comes his way, but he must be able to get it, and his ability to hold his own with men of other professions and in the world of business must ultimately decide whether he shall be in fact, as well as in name, a professional leader in the community, or continue to be regarded by the general public as a sort of an upper class mechanic.

Mr. Blanchard. ARTHUR H. BLANCHARD, M. AM. SOC. C. E.—It is not the speaker's intention to discuss Mr. McCullough's paper from all standpoints, but to call attention briefly to certain phases of the subject which might not be treated in the general discussion.

The speaker wishes to emphasize the author's recommendation that advanced specialized work can be taken profitably by graduate engineers, provided the period of attendance and other details are arranged satisfactorily. Up to this date, very few examples of educational work conducted along these lines are at hand. One case, however, which is conducted on the plan proposed, is that of the graduate courses in Highway Engineering at Columbia University. The period in which these courses are offered is from December 1st to April 1st. Hence an engineer desiring to take all the graduate courses in highway engineering and allied subjects, which fulfill the requirements for the Master's Degree, will necessarily be in attendance for two winter periods, the equivalent of one collegiate year. Although candidacy for the Master's Degree requires as a prerequisite a Bachelor's Degree, nevertheless, mature men are admitted to any courses for which they are qualified, and may take any number of courses.

As this plan is somewhat of an innovation in engineering education, it may be of interest to cite certain facts in connection with the attendance during the winter period of 1911-12, which was the first period under this plan. Although the graduate courses were not brought to the attention of engineers until November, 1911, there were in attendance fifteen men affiliated with highway work, thirteen of whom registered as candidates for the Master's Degree. It is of interest to note that this group included men connected with State highway departments, contractors' organizations, municipal departments, engineering-sales departments of manufacturing companies, county highway departments, and consulting engineers' offices. The experience of these men ranged from one to twelve years. They came from widely distributed localities, Massachusetts, New York, Pennsylvania, Maryland, North Carolina, Alabama, Panama, and British Columbia being represented. **Mr. Blanchard.**

The idea, as suggested by Mr. McCullough, that men taking advanced courses should work on special problems is followed out at Columbia, and it is of interest to note that the founding of several research fellowships by various manufacturing companies is under consideration. The research workers holding these fellowships will investigate problems of particular interest and value to the manufacturing concerns founding them. It is expected that many problems of wide interest to those engaged in highway work will be thoroughly investigated through this medium.

The speaker hopes that the author will elucidate his remarks relative to the injection of an office atmosphere into the classroom. Does the following plan, adopted in connection with the graduate courses in highway engineering at Columbia, approach Mr. McCullough's ideal? This plan consists in the employment of a large number of experts in various fields connected with highway work to act as Non-Resident Lecturers in Highway Engineering. These lecturers cover certain subjects with which they are particularly familiar and their topics form an integral part of the various courses. Although the regular officers of instruction are actively connected with highway work or allied subjects, it was thought that lectures, based on the plan outlined, would tend to broaden the viewpoint of the graduate students, besides bringing them in contact with men of the highest standing in this branch of the Profession.

Mr. McCullough evidently does not fully appreciate the value of a training in French and German. He considers this subject from two standpoints: first, ability to converse in a foreign language; and, second, ability to read foreign literature. The speaker thoroughly agrees with the author in his implied criticism of the time wasted, both in preparatory and technical schools, in the attempt to acquire

Mr. Blanchard.

the ability to converse in French and German. He feels, however, that an entirely wrong impression is given when it is intimated that, for those who have never taken French or German, only a few weeks' work is necessary with a phonograph or in special schools in order to acquire ability to transact business or discuss engineering problems with those speaking a foreign language. Based on the speaker's experience with the use of foreign languages in Europe, and his knowledge of the methods used in teaching French and German in preparatory and technical schools, the following recommendation is offered for consideration: In all foreign language courses for engineers the entire time should be devoted to a thorough study of grammar and to translations. The time now devoted to the reading of French and German in the original is generally wasted. In many cases the pronunciation used by American teachers is poor, and hence those who attempt later to converse in foreign languages must forget the faulty pronunciation acquired previously. An engineer who is called on to use French or German in Europe will find it profitable, after mastering the vocabulary covering his particular field of work, to devote the requisite time to association with a French or German teacher and to living with a family where only the foreign language is used, in order to acquire the native pronunciation and have an opportunity to converse in the foreign language.

The author uses the common argument that "everything of value appearing in the foreign papers is quickly translated." Naturally, the deduction is that engineering literature of value to American engineers is translated and reprinted as it appears in the foreign press. In the field of highway engineering, such is certainly not the case. Before devoting a year to the investigation of the construction and maintenance of roads and pavements in foreign countries, the speaker attempted to review thoroughly the practice of the leading countries of Europe. It was found, however, that the so-called translations referred to gave a very inadequate idea of current practice in foreign countries. The result of the speaker's investigations showed that European engineers had adopted many methods, in connection with the construction and maintenance of highways, with which American engineers were not familiar, and likewise that the few references to this practice in the English press gave a perverted view of foreign practice. That American engineers in many fields may profit materially by thorough study of foreign practice does not require extended argument. Many instances in highway engineering have occurred in which both failures and successes of foreign engineers have been duplicated as experimental work in the United States where such work would not have been undertaken if the experimenters had been familiar with the results of foreign practice. The speaker has in mind an

experiment described by an American engineer, and labeled as a new invention, which had been in use for a number of years in Great Britain, Germany, Austria, and France, and had been described in foreign periodicals. The practice in highway engineering in English speaking countries is very well covered by the technical press of the United States, Canada, and England, but it is the exception to find the best articles printed in the *Annales des Ponts et Chaussées, Annales des Chemins Vicinaux,* and *Le Génie Civil,* of France; the *Annales des Travaux Publics de Belgique;* the *Zeitschrift für Transportwesen und Strassenbau,* and *Der Strassenbau,* of Germany, translated and reprinted or abstracted in the technical press of America. Mr. Blanchard.

PHILIP W. HENRY, M. AM. SOC. C. E.—More or less has been said about education in different branches of engineering, as if it made considerable difference in a man's career whether he takes a course in mechanical, mining, electrical, or civil engineering. It is difficult to differentiate these courses, and the speaker does not think it is necessary to do so. It is the quality of instruction that counts, rather than the subject. A course in mining engineering, properly given, will better fit a man to be a mechanical engineer, than a course in mechanical engineering improperly given. The degree which a man obtains on Commencement Day does not make him an engineer, but indicates, or should indicate, that he knows how to work intelligently on any engineering problem which is set before him. In the class-room he has been compelled, every day of his four years' course, to concentrate his attention on a definite problem, and demonstrate its solution on the blackboard or in some other concrete way. When, after graduation, he takes a position, no matter how humble or in what branch of engineering, he still finds that there is a daily problem to solve, and that, through his training in proper methods of application, he is able to solve it more easily, and thus advance more rapidly than a man, who, with the same mental endowments, has not had the advantage of the same kind of training. In addition to this mental training, good for any kind of business—dry goods or otherwise—the graduate engineer has the advantage of knowing where to go for any detailed technical information bearing on the subject in hand. Mr. Henry.

Many students in engineering schools have only sufficient means to carry them through the course, and, of necessity, must accept the first position open to them. If, therefore, a man who has taken the course of mechanical engineering finds that the only opening is in the office of an engineer whose specialty is sewer construction, he should not despair, but should take that or any other position which may offer advancement, feeling confident that his training will come into use and that he will have the advantage over all

Mr. Henry. his competitors in his ability to work thoroughly and intelligently. By steady application and by taking an interest in his daily task, he will find advancement sure, even though it may not be in that branch of engineering for which he originally prepared himself.

Mr. Rogge. JOHN C. L. ROGGE, M. AM. SOC. C. E.—Professor Swain has stated that when one is studying engineering, he cannot tell what business he will follow ultimately. The speaker would like to say a word or two in reference to engineers engaged in lines of business other than engineering, and to show how circumstances alter cases, using his own career as an example.

He was educated as an engineer and followed the Profession for about twelve years. During part of this time he was employed in one of the New York City Departments where he rose to be Chief Engineer. While thus employed, he was so impressed with the success of various contractors who worked under his supervision and who had little or no education, that when the opportunity came, he resigned his position and entered the business world. The venture was a success, and he has never regretted the change.

While a man's environment, opportunity, and temperament are always large factors in his success, the speaker believes that an engineering education would not be found to be a handicap in any business or profession, because it trains one to reason, to plan, to be keen in observing, to be able to make quick and accurate decisions, and not to take anything for granted, all of which are valuable to one who is in commercial life. A prominent New York lawyer, who was graduated from Stevens Institute as a mechanical engineer and subsequently took up law as a profession, informed the speaker recently that his engineering education had been of great benefit to him in the study of law.

A man who has followed the Engineering Profession for a considerable length of time, however, is apt to be timid as compared with the every-day business man, because of the extreme accuracy demanded by engineering work; but if he will follow engineering just long enough to learn to apply what he has studied in practice, he will then be ready to take up any other line of work or business which may suit him better, or in which there are more financial returns.

To young men studying engineering the speaker would say that there are many opportunities in the commercial world where an engineering education can be used with profit.

Mr. Higgins. CHARLES H. HIGGINS, M. AM. SOC. C. E. (by letter).—This paper is very interesting, expressing as it does, a natural and not uncommon point of view toward this vitally important subject.

The author appears to take for his premises the following: "The engineer should merely give to the teacher his specifications for a good

assistant, and the teacher should try to follow the specifications." For those who accept the foregoing, it can only be a matter of deep regret that the author did not furnish a sample copy of the specifications, including a form of contract and a notice to bidders. The brief description contained in the paper, can, in no wise, alleviate the disappointment felt in not finding the proposed specifications for the finished product. Mr. Higgins.

The author states that "it should not be a difficult matter for teachers to standardize a course of instruction in engineering"; but is it not a little too much to expect of those "whose sole function in life is to prepare assistants for the engineer, and train those who in the future will be engineers," before they receive copies of "a specification for a good assistant" and know the conditions to be imposed by the contract? To illustrate: Some forms of contract contain a clause providing for liquidated damages to the amount of $100 per day for failure to complete the work within the specified time, in full accordance with the specifications. The contracting teacher would have to take such a clause into account in preparing his bid and planning his future course. In all fairness, a copy of the specifications should be sent before the method of carrying on the contract is required.

Discipline and specialization, of course, are good, but is it not a little severe to prescribe, even for teachers, a "sole function in life"? The writer would not be quite so severe; he thinks that he would allow the exercising of at least one more function, even in the case of a hardened offender.

Many engineers not only receive their assistants from colleges, but they send their sons to them, and that gives another point of view.

There is much in the latter part of the paper which the writer would like to endorse heartily, particularly the advantage to be gained in arranging the course so that a man will have obtained some training that will serve to recommend him for a position in engineering work during the summer vacation following the freshman year. Also, the recognition, in the reference to 6 universities and 200 technical schools, of the fact that there may be a distinction; and, above all, the emphasis laid on the importance of a training in English, including public speaking, and in economics.

Perhaps engineers expect too much as assistants, of young men just out of college. Professors of engineering probably know the difficulties of training in college, just as practicing engineers do of continuing that training later in the office. Should the student be trained in details as suggested, it may very well be that he will not detail any steelwork for several years after leaving college; meanwhile, methods of detailing will have changed, or he will find that the office he enters has methods which he must learn to follow.

Mr. Higgins.

Is it the function of the college to take the place of office and field training? The writer thinks not. What it can do is to educate its students in the underlying principles of Nature, and broadly, in the methods of their application, for the use and convenience of Man; and make him more receptive to experiences and capable of interpreting them in the light of the known laws of Nature.

After all, there are distinctions between skill, knowledge, and education. The training which makes the best assistant during the first year out of college is not by any means of necessity the best for the recipient. The college may owe something to the practicing engineer, but it certainly owes vastly more to the students and their parents. The human element will always remain. After all is said and done, engineering is for men and not men for engineering.

Mr. Buerger.

CHARLES B. BUERGER, ASSOC. M. AM. SOC. C. E. (by letter.)—Mr. Green has stated that the aim of immediate usefulness to the future employer may properly be made a secondary consideration in the determination of the curriculum; and it is quite likely that this aim of early usefulness would fail. A course, or a student's electives, may be intended to fit him for a particular position, such as assistant to a consulting engineer, and such position he may never have occasion to fill. Outside of domestic servants, the employé in a subordinate capacity is far from being a free agent, with "liberty" to contract, the Court of Appeals notwithstanding, his occupation being rather a matter of accident than of his wishes or qualifications.

The best curriculum is the broadest one; one which of itself will fit the student for no special position, but will give him the capacity to learn most readily the duties of any one of many possible positions; and his practical education will be obtained, as Mr. Green points out, after he has left school.

Mr. McCullough has not dwelt on the method of teaching, and that is a feature which a teacher should be best qualified to decide; but any one who has been a student has a right to a small voice in the matter. As a rule, the teaching system now comprises 8 months of study per year, 20 hours per week, the time being divided approximately between lectures, quizzes, and the laboratory, the last including shop, field, experimental, testing, and drafting work. In addition, students are expected to put in from 4 to 5 hours each day in private study. The writer would substitute a school year of 50 weeks, with 44 hours of study per week, say 8 hours each for 5 days, and 4 hours on Saturday. He would abolish all lectures and all quizzes, leaving only the laboratory work and the examinations of the present system.

Of the college men with whom the writer came in contact during their student days, numbering, perhaps, 500, four-fifths went through the prescribed courses in a perfunctory way, regarding them as necessary evils, the solace being the shortness of the school hours, and the

time available for other things. Friends of these students in the commercial world were spoken of as being at work; the students themselves were at college, never at work in college. These are only words, but they represent correctly the student's point of view. Mr. Buerger.

The boy of 16 who goes into the business world puts in 8 hours at his daily task, and be he clerk, mill hand, or rivet boy, he takes this length of time as a matter of course. It does not occur to his employer that the boy should work 4 hours a day at his shop, or office, and do the additional 4 hours' work at his own home, should it be work that could be done at home. If the employer did this, he would get exactly as much done in the 4 hours at home as the college student does in his home study. Nor could this boy get 4 months' vacation a year, even without pay, for the employer would consider steadiness of application a primary qualification.

This lengthening of school hours and elimination of home study has not the same meaning as the recent changes in the New York public school system, which have eliminated in effect any study of any kind on the part of the pupil. It is, in fact, the reverse. The study time is moved into the school hours, and these school hours are doubled thereby. The study time is made an essential part of the course; it is even made the only essential, and replaces entirely all lectures and quizzes which, at present, occupy the greater part of the school hours.

This teaching method, then, consists of, say, 32 hours of study per week under supervision, and 12 hours of the various courses belonging under what has been called laboratory work.

The ordinary school lecture is an abomination. In the Stone Age, it was no doubt a proper means of teaching; now there is no excuse for it. It is true that many instructors cannot find books which they consider suitable. With their judgment, the writer will not quarrel; but, even then, they question the value of their lectures by giving the students the substance in mimeographed sheets.

The ordinary quiz is a useful means of teaching the instructor what the student knows, but it is no help in teaching the student what he does not know, and that is what he is after, always granted that there are some capable teachers who make a success of these methods.

Studying under supervision means necessarily individual instruction. This would mean a larger number of instructors, except that it is entirely feasible to use the more efficient students as aids to the instructor to assist the less efficient ones. It would be better, also, to change the terms to correspond to the change in method, and say that the student instructs himself from his printed matter and that he has a supervisor to render necessary aid.

The writer thinks that further elaboration is unnecessary; it can be expressed in two sentences:

Mr. Buerger.

1.—Make the student put in a full day's work every day, and watch him so that he does it.

2.—Apply correspondence-school methods to the college, with the additional advantage of personal contact and personal help.

Such a system will make the student, not a passive receiver, but an active studier, and when he is that, there will be little complaint as to his curriculum.

Mr. McCullough.

ERNEST McCULLOUGH, M. AM. SOC. C. E. (by letter.)—As a teacher, Mr. Garver feels that the writer has presented a paper criticizing teachers, whereas the intention was to assist them in engineering schools by giving suggestions for the better preparation of embryo engineers. The attitude of mind often warps judgment, and, as Mr. Garver read things into the paper that were not there, the writer would suggest that he read it again. For his information, it may be stated that in the Michigan Mining College, Houghton, Mich., the University of Chicago, Chicago, Ill., and Valparaiso University, Valparaiso, Ind., the system of 12-week terms, with new classes in every subject beginning with each term, has been in use for many years. The writer fails to see that these schools have a larger proportion of teachers to students than other schools. The professors have to work a little harder than the majority of professors, almost as hard, in fact, as the majority of engineers in active practice, when the latter are fortunate enough to have a job. The writer understands that a number of private schools also have their doors open throughout the year, and the proportion of teachers to pupils is about the average.

Captain Pillsbury is a graduate of, and has been a teacher in, the finest vocational school in the world. The students are selected after a very careful and severe physical examination followed by a no less severe mental examination. Their conduct is rigidly guided throughout four years of as strenuous work as men can do and survive. This training, however, is in preparation for a position guaranteed to all graduates. A man is even paid while learning. A few years after he has reached his prime, and long before he has outlived his usefulness, he is retired on a pension which, to many engineers in private life, looks like affluence. Criticism made by a man trained under such a system is not as valuable as it might be, for he knows nothing of the trials and tribulations of the average engineer, so long and humorously referred to as a "job chaser." The average student of technical schools has to go through school on very short allowance, and many have to earn the money. On his graduation, no kind Government engages his services. He must strive hard to get a position, and must compete with men having less schooling and more practical experience. The competition is becoming more keen each year. The following* illustrates this point:

* Extract from an article by Edgar Marburg, M. Am. Soc. C. E., entitled, "Engineering Graduates and the World," *Engineering News*, July 4th, 1912.

"It may be of interest to add, that of the total number of graduates, 1 258, beginning with the class of 1873, more than one-half have graduated since 1904." Mr. McCullough.

The graduates referred to are from the Engineering Department of the University of Pennsylvania. The writer has obtained printed matter from other schools, and a study of the subject shows that the foregoing fact is true of the majority of engineering schools. There is no reason for such an increase except widespread advertising, and, in the paper, an endeavor was made to point out a way of altering the present sequence of studies, in order that there might be a continuous elimination of the unfit, beginning with the first year in school. The writer is sorry he failed to make his meaning clear.

The writer also fails to understand where his critics gain the impression that he advocates less mathematics than the present curricula provide. He said "Either mathematics should be taught in a manner that will provide the student with a useful tool, or the time should be given to some other subject." He did not decry the value of a rigorous course in pure mathematics, but he did criticize the slipshod manner in which the subject is taught in too many schools. However, as the question has been raised, it may be said that many eminent educators have stated lately that too much emphasis has been laid on the value of mathematics as a cultural study. That study develops only the mathematical portion of the brain. It does not tend to broaden the mind, and therefore, should be taught rigorously only to those persons who may be apt to require it in later life. It is more difficult to remember than language, and for those who have no mathematical bent it is time wasted to teach anything more than high school mathematics, purely for cultural purposes. The writer fails to see why a "practical" course cannot be "rigorous," and would recommend to his critic a perusal of the book referred to in the paper.

Mr. Constant's discussion meets with the writer's approval. He has evidently read the paper carefully, and it is thought that he must have been in far better touch with actual conditions than the majority of teachers in engineering schools. He goes to the heart of the matter in the following paragraph:

"After all, however, it is not so much the precise nature of the curriculum as the manner in which the subjects and the students are handled that is important. How to bring out the very best in every man, to stimulate his interest and devotion to his work, and, at the same time, to eliminate the lifeless and the small group of deficients always to be found at the lower limit, who, by sheer persistence, in point of time, finally get through, no more fit, perhaps, at the end than at the beginning—this is the real problem of the engineering school."

Compare the foregoing with the last three lines of the second paragraph of the paper.

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For many years the writer tried to get into teaching work, but for three reasons was unable to do so: First, he had never been a teacher in a school of university grade; second, he might have received minor appointments carrying considerably less pay than he could average in practice; third, he was voted against by the faculty in four institutions because the professors said their experience with teachers having many years of practical experience was as a rule unhappy. The man of more than ten years' practical experience does not mix well with the average faculty man. The result is an emulsion rather than a mixture.

Consequently, the writer has been compelled to satisfy his desire to teach, in part, by conducting classes in vocational subjects in institutions to be found in most large cities.

Few teachers in engineering schools are there from deliberate choice. Too many have entered the work because a teaching position was open at a time when they were out of a job. They took the low pay of an instructor to tide them over a winter, and ended by staying permanently. A large part of a teacher's work consists of lecturing, and few men are harder to listen to than the average teacher in an engineering school. A friend once said of a widely advertised professor, "I never listened to a man so reluctant to part with his conversation." The students who had to sit in his classes said of him that he lacked tact, and was so difficult to follow that they failed to see why he was kept year after year. The writer believes there is a far larger proportion of unfit teachers than of unfit men in any industry. Is it any wonder that a man like Mr. Taylor should prepare a paper entitled "Why Manufacturers Dislike College Graduates?" The writer thanks Mr. Constant for his conscientious discussion.

Mr. Green's discussion reads like a high school thesis, and does not contain a single original thought. All he wrote has been written before, and the writer has read such things in discussions on engineering education printed two or more generations ago. This is not a new discussion, by any means, neither can any one put forth really original ideas on the subject. He can only voice the ideas of groups he voluntarily seeks to represent, to the end that there may be improvement. "Qualities make up education, not knowledge." How often that idea is expressed in different words. Lately, some big business man said "I find it is not so much what a man knows, as how he knows it, and character coupled with opportunity, rather than knowledge, determines success and failure." Life is one-half opportunity, one-third ability, and one-sixth technical knowledge as Mr. Green and other young men graduated as engineers will discover sooner or later. It is easily possible to give too much scientific and technical instruction to some young men who would have been served if sent out earlier with somewhat less education, as education is defined in the usual

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academic sense. Mr. Green insists on the duty of the employer to educate the engineers he employs. Does he not know that this is precisely what every employer does; and it is also very costly education. The ultimate consumer pays for it. The writer insists, as the result of twenty-five years' experience since leaving school, that the main object of the majority of engineering schools is to train young men to be competent assistants, and, if blessed by opportunity and backed by ability, they may develop into engineers. First, we must define an engineer, and an attempt to do this was made in the opening paragraph of the paper.

The writer has interviewed every man whom he found willing to talk, and in this way has obtained the opinions and ideas of many hundreds. The majority took up engineering because they wanted a college education, and their parents were willing to give it to them provided they studied engineering, which popularly is supposed to be very lucrative. The prevailing opinion is shown by the effect the Panama Canal had on the enrollment in engineering schools in 1900, many people trying to have their boys graduated in time to secure a position on that work when it would start, in 1903 or 1904. The writer has been told by forty-seven young engineers who were graduated about that time that this was their sole reason for studying engineering. Contractors and other employers do not take engineers fresh from the schools; they take minor assistants. In fact, the fresh graduates usually have a hard time securing employment, few men caring to give them the necessary experience. They must take clerical work, or anything they can get, and then depend on their native ability to go up. They are, in effect, educated by the employer; not as the bricklayer is trained, because there are few bricklaying schools. When trade schools become as relatively plentiful as engineering schools, the large employers will discontinue whatever instructional courses they are now presumed to have, although, in his knowledge of such courses, it is admitted that Mr. Green seems to possess more information than the writer. The writer asks Mr. Green to read carefully the title of the paper and the third page. Engineering education was not therein dealt with as a training in pure or applied science. The title is "Engineering Education in its Relation to Training for Engineering Work," therefore, education was discussed purely from the vocational standpoint.

Mr. Saurbrey, in his opening paragraph, takes occasion to mention the difference between "Engineering Education" and "Engineering Training." A teacher of business once said "When writing a telegram, use no punctuation marks. Hand it to a stranger to read, and if he gets your meaning then send it. If he does not get your meaning, re-write it; but remember, no punctuation." One often neglects to write so clearly that he can be free from criticism by men

Mr. McCullough. who split hairs and hold rigorously to definitions. Mr. Saurbrey objects to the use of the word "pose." As he understands that word, the writer was unfortunate in using it, and perhaps might have said: "The well-read man is generally able to pass as a cultured man." To some, the use of the word "pass," in this connection, still bears too strong a resemblance to a game of poker, therefore is again "pose." Mr. Saurbrey has dilated too much on the unfortunate selection of that word. The writer meant to say that the man who reads deliberately from choice, instead of having manufactured learning stuffed into him by teachers, generally makes the best impression on people who look on the possession of real knowledge as being an evidence of culture. The "poser" was the last man in his mind when he penned the unfortunate sentence. Mr. Saurbrey goes afield, however, in leaving the technical school and going back to the home and the common schools. The writer insists that the technical and engineering schools take the raw material as it is delivered, and, from the first day of school, begin to put in motion a proper law of selection; that and nothing more. His curriculum is practically that of all technical schools of to-day. His arrangement, however, departs from the common one for the purpose of assisting in the early elimination of the unfit, and the dilation of the sense of perception on the part of those who took up the work ignorantly and have in them the germs of engineering ability. A liberal offering of electives gives every man full opportunity to travel as far as he likes in the paths of the scholar, nay, even in the path of the dilettante in matters bookish. Those who like more mathematics than is required can indulge their taste. Those who hanker for the ability to read foreign languages can have their hankerings satisfied.

Mr. Cohen has made a real contribution to the discussion, and is pretty well in accord with the writer in his ideas on the subject, as specifically dealt with according to the title of the paper. Mr. Stengel seemingly has some difficulty in getting at fundamentals. The writer believes that, when a young man is shown how to do a thing and then, in the course of his studies, is given the reason, he is far more likely to take an interest in his work than if he is given a two years' dose of "why" before getting at the "how." The writer, in handling his classes, obtains the best results by training men in doing things, and then giving the reasons when some curiosity is excited. Take the planimeter for example: It was required by a higher instructor that the pupils give the mathematical theory of the planimeter in an examination. The writer first taught the use of the planimeter, and areas were found by it. Then he bent a wire before the class and made a hatchet planimeter. With this crude instrument areas were measured with an accuracy that was surprising. After this preliminary treatment, the elucidation of the theory and the presentation of the funda-

mental equations involved no work, but was attacked with zest. However, to this day, the writer cannot see what difference it made, for the instrument is a commercial product and no engineer is going to make one, unless it be the hatchet planimeter in its crudest form; and then he does not have to know the theory.

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The writer is pleased to learn of the work being done at Columbia University, as described by Mr. Blanchard. The injection of an office atmosphere in graduate courses is well attended to by the method adopted by Mr. Blanchard when it is considered that every man taking the course has had undergraduate instruction, and, subsequently, considerable practical experience. Such men, however, do not require the office atmosphere, because they understand the conditions of engineering life. They really are after the academic side. The office atmosphere mentioned by the writer is something which the undergraduate should breathe from the first, in an engineering school. It cannot be imparted properly when "inbreeding" is the rule in selecting members of the faculty. No man should be employed as an instructor in an engineering school until he has had not less than five years' practical experience of a good character. No graduate of the school should be appointed an instructor, for there are plenty of engineering schools turning out fit men. A man should not be an assistant professor until he has served some time as an instructor; and a graduate of the school can be appointed as an assistant professor, provided he has had not less than five years' practical experience and has also served some years as an instructor in some other school.

Willingness to accept a teaching position should not count so much as a proven ability to teach. An engineering teacher should be a fluent and not a hesitating talker, as so many are. He should be interested in his work and in his students. The writer knows some professors who have nothing to do with their students outside the class-room, and these professors are not men of high standing, it being his observation that the higher standing the teacher has as a man the more of a common man he is with his students. Given teachers with practical experience who know the ups and downs of the "job chaser," the proper tinge of office and works atmosphere can properly be left to them. The writer knows what he would do had he the opportunity to conduct an engineering school, but cannot go into details in a paper such as he presented nor in any discussion. If the teacher cannot eliminate a proper amount of academic atmosphere and substitute a wholesome amount of office and works atmosphere, then he belongs in the liberal arts department rather than the engineering department of the school in which he holds a position on the teaching staff.

Mr. Blanchard does not fully understand the writer in his remarks on the teaching of languages. His criticism of language teaching was

Mr. McCullough. similar to his criticism of the teaching of mathematics in the average school. The writer had the usual high school Latin and Greek. He also studied German years ago and later French. Some years after leaving school he obtained a position where a knowledge of Spanish was necessary, so he added that language to his stock. Of all this language work he retains practically nothing, for he has had no occasion in late years to make use of it. He can read articles in any three of the modern languages mentioned, by keeping a dictionary close to his elbow, and he does a little reading in this way occasionally. Of conversation he is wholly incapable, except that when going home in the street cars he occasionally enjoys family gossip retailed by Germans who imagine no one in their vicinity understands the tongue. Even his meager knowledge of modern foreign languages is superior to that of 90% of the engineers with whom he comes in contact, hence his criticism of the manner of teaching languages in engineering schools, and his suggestion that this study be elective. The engineers who will really profit by it will take up this work; the "ninety and nine" who go at engineering as a vocation, and not with any idea of the study of engineering as a cultural matter, nor with the idea of being teachers, nor with any idea of doing research work, will not study foreign languages at school from choice, unless the credits gained thereby are more easily obtained than by any other method. The writer's criticism did not extend solely to the waste of time in attempting to get a conversational knowledge of a foreign tongue, but to the very poor way in which, as a rule, the study of foreign languages is taught in the majority of schools to first- and second-year students, who are obliged to take the work. It is really a device for piling up credits.

To a certain extent, the writer agrees with Mr. Boucher on the subject of the 6-year course in engineering schools. He stated in his paper a belief that engineering schools of the future in the United States will probably call for a minimum of 6 years' work. The reason for this belief is that there is a widespread demand on the part of teachers that this be accomplished. The tendency in this direction is so strong that no power on earth can prevent it from being tried. Much of the elementary work now being performed in technical schools of college grade will be attended to in technical high schools, so that in the future we shall have the Trade, the Vocation, the Business, and the Profession of Engineering, all recognized and taken care of in schools ranging from trade and high schools to the largest universities. The greater number of teachers will come from schools where the professional ideal is held, that is, these higher schools will train teachers, many of whom it is to be hoped will have considerable active practice in earning a living as vocational men before taking up teaching. The writer, in his paper, took the vocational school,

corresponding to the present technical schools, as the one in which engineers should be most interested. The present 5- and 6-year courses, however, give very little, if any more, than the 4-year course in some schools, for the latter require from the students more hours per week than schools with the longer courses. Mr. McCullough.

Mr. Boucher also referred to the writer's neglect to include arithmetic as an entrance subject. The writer has taught much in evening schools, and, as a result of his experience, can say that arithmetic is taught so badly in the ordinary American school that it will be better to omit it as an entrance subject, assuming that it was completed before the student entered the high school. His experience as an instructor in evening schools, and also as an employer of office assistants and draftsmen, compels him to say that the schools of America have much to learn from the schools of Europe in teaching arithmetic. It is stated in the paper that in the first year students should devote one hour each day to going through the examples in Sanborn's "Mechanics' Problems." This will give them drill in arithmetic. He mentioned also that the second-year students should be drilled on problems apt to arise every day in actual work, these problems all being arithmetical rather than algebraic.

In reply to Mr. Hunt the writer will say that it is a fact that the "tendency of the modern technical school is to become more and more narrow." This the writer wishes to counteract by his proposed arrangement of the curriculum. It will be noticed that he adheres closely to essentials throughout, merely changing the order of their introduction, with the object of broadening the minds of the men taking the work. The young man is interested in the practical rather than the ideal. He studies engineering in order that he may be enabled to earn a living. It is a mistake to cram his sciences, economics, psychology, etc., down his throat during the years when he does not and cannot appreciate them. He should be given at first the things which will make him most immediately useful to his prospective employer, to the end that the narrow-minded and undeveloped boys will be worked off by stages, leaving those whose minds develop with the school work. The humanities, therefore, come at a time when the student is maturing and the topics of the day begin to interest him. The young boy is intensely egoistic, albeit without knowing himself to be so. At about the time he reaches the age when he can vote, the problems of society begin to interest him; also, at this age, he is, as a rule, unselfish and gregarious. If he now takes up the subjects that interest men and women of standing, they will make an impression on his mind which can never be effaced, and, later, when he achieves success, he will not be considered a sort of upper-class mechanic.

Mr. Henry states that the quality of the instruction counts, rather than the instruction. It is precisely this point that the writer sought

Mr. McCullough. to bring out. He believes that the quality of instruction in the majority of engineering schools can be vastly improved. The essentials have been pretty well settled by a century of teaching. The order and the manner in which these essentials shall be imparted are now matters requiring settlement, bearing in mind that 99% of the students in engineering schools attend these schools for vocational training. When a man is drilled enough in mathematical, physical, and chemical sciences to read intelligently along the lines of his calling, he has obtained a great deal. It has been stated* that "a technical education can do nothing more beneficial for a man than to make him familiar with the best and most authoritative engineering literature." Granting that technical education gives him this much, let us add certain other broadening studies of a general nature, so that the graduate of the engineering school will be a good assistant, a well-read man, a good citizen. Those who leave before graduation will be good minor assistants, whose further development will depend on their inheritance of mentality and family environment.

Mr. Allen will find on investigation that a comparison between engineering, law, and medical schools is not at all unfavorable to engineering schools. He says "nowadays, the majority of law students first acquire a general college education, etc." It would be interesting to know where he obtained the data on which to base this assertion. A majority of the men admitted to practice as attorneys are not graduates of law schools, even to-day. A majority of graduates take courses, of two years in some States and three years in others, in schools run for profit, many of them being schools having evening sessions only. A very small percentage is graduated from schools requiring a college degree for entrance, there being less than half a dozen such schools in the United States, and these have small classes. Eminent lawyers are endeavoring to have entrance requirements stiffened, with a view to eliminating competition. Less than half a dozen medical schools require the completion of a college education before entrance, and perhaps a dozen call for two years of college work after high school. A few years ago there were 176 medical schools in the United States, but last year only 116 were reported, the recent campaign against medical schools run for profit having resulted in good. Medical men, however, are divided on the question of too severe entrance requirements. Eminent physicians and surgeons give long lists of names of men who were instrumental in advancing medical knowledge, and would never have entered the medical profession had they been compelled to complete a 4 years' college course before studying medicine. It has been stated also that few discoveries of importance have been made by men not pressed by poverty, for the temptations to ease are hard to resist when men have the means to gratify their inclinations to loaf.

* *Engineering News*, November 17th, 1910.

The argument is that only men backed by families of means can take a medical course if the entrance requirements are very severe. Mr. McCullough.

The movement to require longer preparation before studying law or medicine is inspired by the desire to cut down the number of practitioners. It is felt by some that, while this may eliminate a few good men, the resulting good to the profession in the improvement of the quality of the majority secured, will compensate for such possible loss. Opponents of the proposition point out that the loss possibly of another Jenner, or Harvey, or Lister is a large price to pay for securing an increased number of men fitted to shine socially, for the additional education required is not medical or surgical, but merely cultural, to the end that the members of the profession may make a good showing at "pink teas." Similar ideas prevail among men in the Engineering Profession. Some hope to have 6-year courses common, because, "there are too many engineers." Some wish to have two additional years for the purpose of enabling engineers to shine to better advantage socially. Some want a 4-year college course completed before beginning the study of engineering, for the same reason. At all events, it is seldom that the additional 2, 3, or 4 years are presumed to be spent on engineering subjects. It is pretty well settled that 3 or 4 years will suffice for the vocational studies connected with engineering, and the additional years are to be spent on the study of subjects of general interest. The writer proposes a re-adjustment of the curriculum, so that the general subjects may well come in the final years, the student being put at the vocational work as soon as possible.

Mr. Allen says "the schools will furnish that kind of education for which there is a strong demand from the students themselves." This is very pretty, but the truth is that few, if any, students entering engineering schools know what they need, still less what they want. Skilful advertising can make them believe they want anything the advertising department of the school presents for their attention. The students, that is, the undergraduates, should have nothing to say about what they want. Those who go in for vocational studies should get them. Those who can afford to wait until the completion of a college course can do so, but the fact remains that whatever road they take to obtain a degree in engineering, on graduation they must "hunt a job." The training offered at an engineering school should be such that the graduates will be enabled to fit in quickly, wherever employed. It is known that graduates of engineering schools may look confidently forward to salaried employment shortly after graduation, whereas graduates of law and medical schools generally contemplate going into business for themselves. Their training is of an eminently practical nature. The law schools have moot courts and also require a certain amount of time to be spent in court, in the search

Mr. McCullough. for precedents, and the study of famous cases in libraries. The lecturers are nearly all eminent attorneys who lecture on their specialties. Such lectures are not as technical as lectures of engineers and are usually a guide to the critical study of some text. Medical students begin early on dissection of bodies, and from the first attend clinics in the college and assist in operations. The lecturers in the medical schools are also surgeons and physicians of standing, whose lectures are expository and non-technical guides to the critical study of texts. Lawyers, physicians, and surgeons, as well as ministers of the Gospel (whose divinity schools are vocational schools of an extreme type) are considered to be well educated, cultured men because they mingle daily with people who are well read and cultured, and cannot fail to obtain a certain degree of polish. They also have plenty of time to do considerable reading of a general character, and can discuss intelligently the questions of the day.

Law and medical students are not ignorant of conditions to be encountered in the practice of their respective professions. They go valiantly into the fight for existence, hoping to succeed and willing to stay as long as they have any staying powers. Engineering students as a rule are inexpressibly shocked after graduation when they come face to face with conditions of employment and compensation. They believe, on entering school, that the Profession is most remunerative. They find after graduation that steady positions are the exception, and that pay does not invariably increase with years of experience and increased ability. They cannot go into private practice until near middle age and after the acquirement of considerable general experience. The variety of work performed by engineers during 20 years is remarkable when one makes a study of the lives of engineers, as shown by the biographies printed in the *Transactions* of this Society. Their training as engineers is received after leaving school. The training in school is to enable them to acquire quickly, and with certainty, much that they might acquire in a practical way in offices, with the expenditure of considerably more time and energy. That is, school training for engineers is an efficiency proposition, to enable them early to be of service to their employers and of value to themselves, to the end that they may sooner mount the lower steps on the ladder of success and be engaged on work of high grade while still young and full of energy—not yet discouraged and weary because of the hard battle of life. If the application of their studies to the practical problems of their life work is taught them early at school many will secure positions with the start given in the first one or two years in school and not remain to be graduated, while others will certainly stay to get more at school.

The writer is not opposed to embryo engineers remaining 10 years in school if they wish, nor to engineers stringing an alphabet of honors

after their names, representing degrees conferred in course. He will gladly welcome the day when the general public looks on engineers as being at least as well educated as men belonging to what have heretofore been termed "the learned professions." In fact, he is not certain that the day has not arrived, for engineering at present is popularly supposed to be most desirable as a profession and business, the average man looking on engineers as men who have pursued a hard course of study in school, practical, but scientific. The writer, however, is opposed to the idea that all engineering students must receive their education in the same way, and in the same number of years, regardless of ability, or inherited, or acquired characteristics. The true engineer is a student all his life, the technical school giving him merely a start. We cannot compare methods in schools for other vocations with methods in engineering schools, for in law, medicine, and theology, one path in each must be followed, while engineering is a profession to which many distinct trades contribute. Mr. McCullough.

Mr. Rogge well illustrates one point the writer might have brought out. The tendency among too many engineers is to magnify unduly the scientific and the clerical, or, as they term it, the technical, side of the work. Mr. Rogge saw that greater opportunities existed for him in getting into the business side of engineering, success following very quickly. He utilized his engineering education. It is more than likely that if he had spent several years more in school his sense of proportion would have been altered, and he would have stayed with the office instead of going out into the field as a business man.

The writer has a good friend, a consulting engineer of wide reputation, who is termed, by envious engineers, "a bluffer." There is not the slightest doubt that he would fail signally as an engineer, in the sense considered by the majority of the men contributing to this discussion, but, as an adviser on engineering matters, he is good. He was asked how he came to be so successful and said:

"The school I attended treated me badly in the way of an education, and I figured after a couple of years' work that I was doomed to be a failure in the designing end, so I took a job as timekeeper and gradually worked up until I got into business for myself as a contractor. When I failed, and failed so big that my case attracted the attention of newspapers, I found myself in demand as a practical man to advise on big construction matters, and now I am a consulting engineer and making more money than any man in my class."

The writer knows another man who also failed to get at school what he had hoped for, but who, by self study, has finally acquired all that other men received in technical schools. His success has not been marked, because he looked too much on the clerical end of engineering as the main thing, instead of looking on his education as being merely preparatory to his entrance on life.

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Some time ago, the writer received a visit from a friend who lately resigned a position, in which he received a good salary, in order that he might go into private practice. He now regrets the action. On being asked why he left, he replied that his employers kept crowding so much of their general work on him that he had no time to attend to his engineering duties, and had to leave them to young assistants. He was disgusted at having to take up many legal points and at having to bother with contractors and their troubles. His idea of engineering was to design structures. The tendency is marked among men who put many years in school to assume just this attitude, and the logical place for such men is the school room as teachers, after they have obtained some practical experience. The writer believes, and has many times expressed in writing his belief, that an engineering course is the modern ideal in education, as opposed to the classical course. It will hurt no one to take such a course, provided he can always understand that every man who takes it should not do so with the idea of being a professional engineer. As a preliminary training for business life, it ranks with a legal education. The writer likes Mr. Rogge's discussion.

In regard to the remarks of Mr. Higgins, the writer feels it necessary again to call attention to the fact that he merely proposed a re-arrangement of the curricula of technical schools so that boys with low ideals might sooner be fit to leave and go to work. Let each student feel each year that he is a little better prepared to earn a living, and if he stops going to school before he has done all the work required for a degree, he may be doing the best thing for himself and the best thing for the Profession. When the ups and downs of engineers are as well known to the general public as are the trials and tribulations of lawyers, medical men, and ministers, so that all young men who go to engineering schools face their future with wide open eyes, such discussions as this will be out of date. The writer distinctly referred to the fact that his paper is intended to deal with the technical schools of the present day, not the university engineering schools of the future, when what is exceptional knowledge now will then be common knowledge. It is fascinating to think of what our great-grandchildren may have to master before they will be considered fit to practice a profession, or even earn a living.

Whatever Professor Swain writes is good to read, and the writer is flattered that he took time to discuss the paper. The writer does not by any means consider it a bad thing that men educated in technical schools often turn to other lines of work, and regrets that it was possible for any one reading his paper to get that impression. He does regret that the courses of study are arranged so that students seldom get to the practical side of their work until the last couple of years, this forcing them to stay in the Profession merely be-

cause they feel that their long training would be wasted. Parents who pay the bills always feel that way, so the courses of study might be arranged to give the young men practical training from the start. It has been asked what specifications an engineer might propose. They have been pretty well stated by Professor Swain: "He wants a man who is faithful, who is of good character, conscientious, who can think straight, who will not be anxious to stop work as soon as the bell rings, who will be loyal to his employer, who has 'gumption,' and who can meet emergencies." He might add that the school should also take considerable pains to make the students understand the actual conditions attached to engineering employment and the compensation therefor, the importance of living on half the pay when earning, to understand that employers have nothing against young graduates as such, but because few of them are worth their small pay for several months after leaving school, some not for a year or more. Employers also want men skilled in common arithmetical computation and with the ability to make neat drawings and do decent lettering; these, in addition to all the qualities of manhood mentioned by Professor Swain and necessary as well in other lines of business. Young men are not intrusted with important work, so their education should fit them to do well the small and comparatively unimportant things their employers put them at. A careful reader of the paper should see that the writer lays considerable stress on the studies enabling men to mix well with the world. Mr. McCullough.

A high standard for the Engineering Profession is very well, and the writer is as keen for it as any engineer, but the paper he presented was from the point of view of the more than 90% of students who take engineering courses for their purely vocational, and not for their cultural, value. These green young men and boys enter a school to study engineering with the intention of earning a living at engineering work, and do not know what it implies or what the real opportunities are. At the end of the freshman year they must select some specialty, still ignorant, for the freshman year is merely an extension of high school and there is seemingly no tie in it to the life of an engineer. A month ago a young man called on the writer for advice as to his future. He entered a State university for a college course and met a boy who persuaded him to enter the college of engineering. This was the first time he knew that engineering did not necessarily mean the running of an engine. He remarked that he could see little difference between the freshman work and the senior year in high school, and drifted along unthinkingly until spring when he was suddenly made aware of the fact that the university gave eleven distinct engineering courses, and he must make a selection of a specialty. He still knew no more about the calling of the engineer than he did on leaving high school. His parents could not help

Mr. McCullough. him, but his indecision was settled by a series of social events of the eleven engineering societies, who were engaged in "rushing" freshmen. The mining society gave what he called the "swellest" reception and entertainment and had the best floats in the annual college parade. The career of John Hays Hammond was at that time attracting considerable newspaper and magazine attention, so the boy entered the mining school. This may sound far fetched, but he states as a fact that he took the sophomore and junior work in the mining department without seeing a mine. A requirement of the school is that students must spend not less than 3 months in some vacation in actual mining work, in order to be eligible to enter the senior class and obtain the degree in mining. In his first vacation he helped the county surveyor near home. In the second vacation he was a draftsman in the office of a structural engineer. This past summer he had to do mining work or be unable to register this fall as a senior, so he went into a mining district to seek employment. He worked for 3 months, but to the last day was unable to rid himself of a disagreeable feeling in the pit of his stomach when going down a shaft. He was always impressed with a feeling of insecurity when in the workings, and the number of accidents he witnessed were not reassuring. On top of the ground he is all right, but he hates to think of spending his life in mines. He was advised to complete his course of study and get rid of the feeling that since he studied mining engineering he must of necessity follow that as a profession. His training in surveying, drafting, mathematics, physics, and chemistry will enable him to be a good assistant in the office of an engineer or manufacturer, which, after all, is the most that a technical school should expect to give, the technical school, it must be remembered, being something different from a high-grade engineering school attached to a university and headed by men like Professor Swain.

Professor Swain says: "It is impossible for a man who has not tried to teach to draw up a curriculum which will work well; he almost always forgets that the problem of engineering education, or of education in general, is not an engineering problem, but a human problem." The writer begs to state that he has not only tried to teach, but is rated as a successful teacher. He has taken classes abandoned by professional teachers, and greatly increased them in number because he understood the men with whom he was dealing, their problems encountered in trying to earn a living, their object in studying at night after working all day, the best methods of handling them so as to inspire interest in the subject and hold it to the completion of the work. He has also been successful in coaching young men unable to follow intelligently their paid teachers in college and technical schools, boys who would otherwise have been "flunkers." In his paper he endeavored to deal with the problem of engineering education as a human problem, the sub-

ject being discussed as a vocational, and not a purely educational proposition. He has tried to suggest that the instruction be imparted somewhat more practically in the first two years, in a human and humane manner. The writer must remind Professor Swain, as he has other men who have presented discussions, that he has merely changed the order of studies and not proposed a brand new curriculum. Mr. McCullough.

In reply to Messrs. Fuller and Buerger, the writer must call their attention—as he has called the attention of others preceding them in the discussion—to the paper. He fails to find anything in it to lead any one to believe that he advocates a narrow training or that he decries education of the proper kind. He simply attempts to rearrange the curriculum, omitting nothing of value, adding much of value, and postponing to the reasoning years subjects deemed "cultural"; leaving the study of economics, history, literature, sociology, etc., to minds capable of reasoning.

Teachers uniformly resent suggestions from practicing engineers and from employers of engineering graduates, claiming that such suggestions have a narrowing tendency, and that men not teachers do not put the proper "cultural" value of education to the front. This is not borne out by the facts. A study of discussions on engineering education, from the time such discussions commenced, will show that the practicing engineer has been more instrumental than the teaching engineer in having more attention paid to general subjects. The practicing engineer laughs at the long array of specialities listed in catalogues of engineering schools, and knows, as the result of actual experience in winning a living, that a few fundamental things well taught are sufficient; but they must be well taught. The teachers, each one anxious to magnify his importance in the faculty and gain glory and higher pay, are the men responsible for the narrowing of the curriculum. Teachers, by pushing special courses, which the bewildered freshman must consider, stultify their remarks about general education and the cultural value of education. Professor Fuller says:

"In talking with some of his own students the speaker has noticed a greater inclination to take general work in the latter part of the curriculum than in the first. If given in the first part, it is thrust upon them; if available later, many will take it willingly. The speaker has heard practicing engineers suggest such an arrangement."

If this be so, then why not try it?

Without wishing to appear to be a critic of teachers, for he also teaches, because he likes it and teaches a class of men who come voluntarily to get the work, the writer must say that no class of men is less tolerant of suggestion and apparent criticism, than teachers, beginning with the kindergarten grade. This is for the reason that teaching is a vast organized profession, fettered with precedent and

Mr. McCullough. hampered by tradition. These remarks must be softened by the statement, that with all the criticism of the teaching class indulged in by people who must employ the product turned out of institutions of learning, the greatest changes and improvements in teaching methods have come from the ranks of the teachers. However, there is a deadening influence at work tending to weaken those who teach continuously many years. For this reason, the writer is greatly in favor of teachers in technical schools being employed on practical work, and thinks there should be a greater amount of practical work demanded of them. Good teachers should be given leave of absence at stated times, under full pay, so that they may go into the ranks of engineers, to the end that the deadly monotony, inherent in all large organizations and classes, shall not stunt their minds.

Professor Fuller asks, with others, for a specification for the preparation of engineers' assistants. It has been given already in this closure, as well as in the paper. The writer nowhere stated, nor did he imply, that the product of the "engineer factories" should be guaranteed, as some of the gentlemen who have discussed the paper facetiously remarked. A reference again to the paper is suggested. The reason for asking that the wishes of the employer be more carefully considered has been sufficiently dealt with in the paper and in this closure.

The writer agrees with Mr. Buerger that the best training is the most broad, and that a division into specialties is to be deplored, as far as undergraduates are concerned. Employers, however, are not willing to give all the practical training so essential. There are too many thousands of graduates turned out annually from technical schools to compel the employer to waste much time with the unfit and incompletely trained. A three-line advertisement in the Sunday edition of any good daily paper will suffice to fill the mail box to overflowing with applications for work. Short shrift is given those who do not take hold quickly. Many who might otherwise have been successful are doomed to wander for many years from job to job, because of the false view of life obtained in the institutions supposed to be created for the purpose of supplying the demand of the industrial world for trained workers. The technical school is assumed to exist for a particular purpose, and it does not fulfill its mission if the majority of graduates fail to meet with as much success as the average man.

The writer endorses most heartily all that Mr. Buerger says, beginning with the words, "The ordinary school lecture is an abomination," and continuing to the end of his discussion, which should be taken to heart by every teacher, every practicing engineer, and every employer of the product of engineering schools. Make the boys work hard from the start. Teach a smaller number of subjects at one time if

necessary, to carry out the ideas expressed in his two sentences relating to methods of teaching. The employment of older students to assist the teacher is excellent, as the writer has found in his own teaching experience, for it helps every one. A man learns best when he has to teach, and the student is inspired when he works with his teacher, instead of trying to do what he is told to do, with occasional guidance from one who assumes a superior attitude.

Mr. McCullough.

A fitting end to this discussion is the following:*

"*Educating the Educators.*—The University of Cincinnati was one of the first in this country to apply continuation school methods—giving a pupil shop practice under actual commercial conditions, along with textual instruction. Dean Schneider, of the engineering college, has made some interesting confessions of the reflex action upon the university faculty of this practical shop training. He says:

"'We learned the first year, and have had it verified each year since, that the shop will spot a yellow streak in a man before the university even suspects it. An attempt to sneak through spoiled work is never a great success there. We, at the college end, soon found our work under scrutiny and criticism from a source that does not hesitate to scrutinize and criticise. We are brought face to face with the failure of a university department as we never are in our four-year courses. A student, let us say, has finished successfully his work in physics. Some day he does a fool thing in the shop which indicates that he knows very little about the subject. When you confront him with the fool thing, and with the fact that he should have known better because he had been taught the theory governing it, you find his grasp upon the theory to be very feeble.'

"Practical education will teach the teachers. We imagine it would not be a bad thing in every university if pupils and instructors, pleasantly loafing through their four-year literary courses, were periodically checked up by some hard-and-fast test drawn from actual life outside the campus, whereby they could discover exactly how efficient their processes were."

* Editorial from *The Saturday Evening Post*, October 5th, 1912.

MEMOIRS OF DECEASED MEMBERS.

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

ALFRED ELLSWORTH CARTER, M. Am. Soc. C. E.*

DIED JUNE 11TH, 1912.

Alfred Ellsworth Carter was born at Blair, Nebr., on April 19th, 1867, of American parents, his ancestry dating back through several generations of pioneer stock. His early education was obtained at the public schools of his home town, and, later, at the University of Nebraska, from which he received the degree of Bachelor of Science, in 1900. In 1902 he entered Columbia University, New York City, and was graduated in 1904 with the degree of Civil Engineer.

Mr. Carter's early experience was gained while earning his way through college. In the the early Nineties he was in the employ of the Chicago and Northwestern Railway Company, in Fremont, Elkhorn and Missouri Valley Railroad activities, holding successively the positions of Chainman, Rodman, and Transitman, on miscellaneous surveys in Nebraska and the Black Hills of South Dakota. From August, 1897, to March, 1899, he was Assistant Engineer on the construction of a hydro-electric power dam at Divide, Mont., an impounding reservoir dam adjacent to Butte, Mont., and one of the first wood stave and riveted steel pipe lines for the Montana Power Company.

Following the Spanish-American War, from October, 1900, to October, 1902, Mr. Carter was Assistant Engineer in charge of detailed designing of sewers and pumping stations for two sections of the marginal sewer system of Havana, Cuba, being associated with Samuel M. Gray, M. Am. Soc. C. E., Consulting Engineer, the work being done by the Department of Sewers, under Military Government, William M. Black, M. Am. Soc. C. E., Colonel, Corps of Engineers, U. S. A., being in general charge at Havana.

From January, 1905, to the time of his death, Mr. Carter was in the employ of the Rapid Transit Subway Construction Company, Contractors, of New York City, as Assistant Engineer, until 1908, in charge of tunnel alignment, check surveys, track-laying, and driving reinforced concrete piles, on the construction of the East River Tunnel of the Rapid Transit Railroad; then Resident Engineer in charge of construction of the Bowling Green Shuttle Station, and the Subway station extensions at the Fulton Street, Wall Street, Bowling Green, Borough Hall, and Atlantic Avenue Stations of the Interborough

* Memoir prepared by George H. Pegram, M. Am. Soc. C. E.

Rapid Transit Company. He was also engaged in reporting on extra claims of the Sub-contractor on the East River Tunnel.

At the time Mr. Carter became engaged on the work of the East River Tunnel, the Brooklyn tubes were just entering the river section and the Manhattan tubes had not emerged from the rock. He was employed continuously on this work until its completion in January, 1908. It has been described as one of the most difficult pieces of engineering work ever accomplished. Mr. Carter's position as Assistant Engineer imposed great responsibilities on him. He was in charge of the delicate operations of sinking piles through the bottom of the tubes to rock. The character of the work and the financial failure of the Sub-contractor, during the construction of the tunnel, made the accounting unusually complicated. The patience and fidelity with which Mr. Carter worked in checking the claims of the Sub-contractor and the skill and judgment evinced in his reports are remarkable. It was a work of great labor and uncongenial to an Engineer, but his familiarity with the construction forced it on him. His engineering work had been above criticism, but this work was almost above praise.

Subsequently, Mr. Carter was put in charge, as Resident Engineer, of the construction of Bowling Green Shuttle Station and the lengthening of the Subway stations from Fulton Street, Manhattan, to Atlantic Avenue, Brooklyn. Like his East River Tunnel experience, this work was of the most difficult character. The continuous operation of trains, the congested street traffic, the numerous sub-surface structures which interfered with the work, such as sewers, water pipes, electric subways, and the foundations of buildings, made it always a delicate task.

The work of extending the Borough Hall Station in Brooklyn, for which Mr. Carter designed the shoring and directed the work for the Construction Company, was especially difficult. This station was built of reinforced concrete which was exceptionally difficult to remove. At this point there are three tracks in the Subway, with cross-overs, and on the surface of the street there is a junction of two tracks in Court Street and two tracks in Fulton Street. Both side-walls of the Subway, for a length of 135 ft., were entirely removed, and its roof with 7 ft. of cover, together with street structures, etc., was supported on timber. Three columns of the Elevated Railroad in Brooklyn were temporarily supported over the work and the foundations renewed; the cast-iron pipes and the gate-chambers of the high-pressure water mains were supported and reconstructed at an especial menace to the work. In addition, the portico of the County Court House, weighing more than 1 000 tons, a structure with four large granite columns, thus having little transverse stiffness, was temporarily supported, and the foundations were carried 12 ft. deeper by masonry

underpinning. This was done without the slightest show of crack or any measurable settlement of the portico. All this work was in sand and Mr. Carter was continually obliged to render it safe against the breakage of water pipes or the unusual flood of storm-water.

This work was about completed at the time of his death, which occurred suddenly at his home in New York City on June 11th, 1912, from cerebral hemorrhage.

Mr. Carter was a man of sterling integrity, with the ability for doing hard work well, and accepting and fulfilling growing responsibilities with quietness and efficiency; the consideration he gave to all matters, large or small, entrusted to his care, had won for him the respect of his associates and those who worked under his direction.

In 1904 Mr. Carter was married to Miss Ida C. Messer, of Cleveland, Ohio, who survives him. She is a lady of unusual educational attainments and was able to assist him in his professional work.

He was a Member of Columbia Chapter (Kappa) of the Society of the Sigma Xi, and a Member of Washington Lodge No. 21, F. & A. M., of Blair, Nebr.

Mr. Carter was elected an Associate Member of the American Society of Civil Engineers, on June 4th, 1902, and a Member on April 4th, 1911.

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Vol. XXXVIII. NOVEMBER, 1912. No. 9.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

IRRIGATION AND RIVER CONTROL IN THE COLORADO RIVER DELTA.

By H. T. Cory, M. Am. Soc. C. E.

To be Presented January 1st, 1913.

From almost every point of view, the Lower Colorado River, and particularly the Colorado Delta, is extremely interesting. Ever since its examination and description by members of Lieut. Williamson's exploration party in 1850, the various features, geological, geographical, anthropological, engineering, and otherwise, have been written about. In 1905 the diversion of the Colorado River into the Salton Sea and the events which followed it were so spectacular as to result in world-wide notoriety.

While engaged in re-diverting the river, the writer became impressed with the fact that the experience and information obtained should be made available to the Engineering Profession, and since then he has constantly been gathering data to that end. In February, 1907, a general paper on the subject* was contributed to this Society by C. E. Grunsky, M. Am. Soc. C. E., then Consulting Engineer to the Secretary of the Interior in United States Reclamation Service matters; so that, before giving detailed information, it seemed best to wait until time should have revealed the strong and weak points

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

* "The Lower Colorado River and the Salton Basin," *Transactions*, Am. Soc. C. E., Vol. LIX, p. 1.

of construction and methods. Since then, experience with the control of the Lower Colorado River, and as local executive head of the immense irrigation project of the Imperial Valley, has brought the conclusion that the various possible vicissitudes of irrigation enterprises in the United States have been so well exemplified in the region as to justify setting forth such experience in considerable detail.

Ordinarily, more information is secured from failure than from success; consequently, no apology should be due for pointing out failures as well as successes in a paper, the functions of which are primarily to furnish useful engineering information.

The Colorado River.

The United States Geological Survey has observed the discharge of the Colorado and its several tributaries since 1895, and the results are to be found in its Annual Reports and later in the Water Supply and Irrigation Papers, especially Nos. 249 and 269, on the Colorado River Basin. At various times 169 gauging stations have been maintained, and there are 76 at present.

General Discharge Characteristics.—From the data obtained at these stations, the discharge characteristics of the tributaries and main Colorado River are pretty well determined. The discharge records of the Green River, at Green River, Utah, the lowest gauging station above its mouth, and where the drainage area above it is 38 200 sq. miles, indicate a maximum flow of about 75 000 sec-ft., a minimum flow of about 700 sec-ft., and an average annual run-off of about 5 000 000 acre-ft. The greatest discharge is in June, averaging about 1 600 000 acre-ft.; the annual rise starts about April 1st, reaches its peak in the middle of June, and has passed by August 1st.

The data obtained on the Grand River indicate a proportionately great run-off and very much the same distribution throughout the year. The records, taken at Turley, N. Mex., on the San Juan River until December, 1908, and since then at Blanco, indicate an ordinary flood maximum of about 15 000 sec-ft., a minimum of 75 sec-ft., and an average annual discharge of 1 000 000 acre-ft., but with a much longer period of summer flood than in the Green and Grand.

The maximum flood discharge of the Little Colorado when it enters the Colorado River is not known, but is probably about 50 000 sec-ft. The floods are short and violent, and carry large quantities of silt in

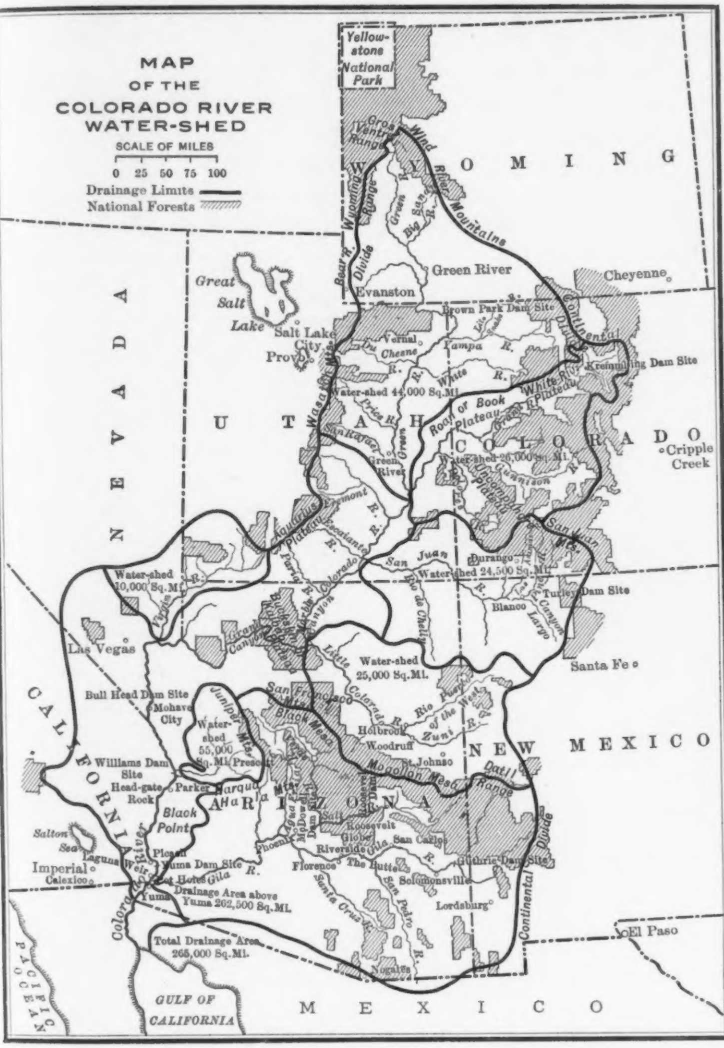
MAP
OF THE
COLORADO RIVER
WATER-SHED
SCALE OF MILES
0 25 50 75 100
Drainage Limits
National Forests
Yellowstone National Park
WYOMING
UTAH
NEVADA
COLORADO
ARIZONA
NEW MEXICO
CALIFORNIA
MEXICO
GULF OF CALIFORNIA
PACIFIC OCEAN
Great Salt Lake
Salt Lake City
Provo
Evanston
Green River
Cheyenne
Brown Park Dam Site
Kremmling Dam Site
Cripple Creek
Water-shed 44,000 Sq. Mi.
Water-shed 26,000 Sq. Mi.
Water-shed 24,500 Sq. Mi.
Water-shed 10,000 Sq. Mi.
Water-shed 25,000 Sq. Mi.
Water-shed 55,000 Sq. Mi.
Durango
Turley Dam Site
Blanco
Santa Fe
Las Vegas
Bull Head Dam Site
Mohave City
Williams Dam Site
Head-gate Rock
Parker
Black Point
Laguna Weir
Yuma Dam Site
Yuma
Drainage Area above Yuma 262,500 Sq. Mi.
Total Drainage Area 265,000 Sq. Mi.
Imperial
Calexico
Salton Sea
Phoenix
Florence
The Butte
Globe
Riverside
San Carlos
Roosevelt
Solomonsville
Guthrie Dam Site
Lordsburg
El Paso
Nogales
Holbrook
Woodruff
St. Johns
Prescott
Continental Divide
Wind River Mountains
Wasatch Mts.
Roan or Book Plateau
Uncompahgre Plateau
San Juan Mts.
Aquarius Plateau
Grand Canyon
Black Mesa
Mogollon Mesa
Datil Range
Juniper Mts.
San Francisco
Little Colorado R.
Rio Puerco of the West
Zuni R.
Gila R.
San Pedro R.
Santa Cruz R.
Colorado River
Green R.
Bear R.
Wyoming Range
Gros Ventre Range
Duchesne R.
White R.
Yampa R.
Price R.
San Rafael
Fremont R.
Escalante R.
Paria R.
Virgin R.
San Juan R.
Rio de Chelly
Gunnison R.
Grand R.

Fig. 1.

suspension, in which regard the stream is similar to the Gila and Salt Rivers.

The Gila at Yuma is often dry, and has a maximum flashy flood discharge of probably 185 000 sec-ft. with a total average annual run-off of 2 750 000 acre-ft. Flashy floods have been known to occur in every month of the year except May, June, and July, at which times the Colorado has its maximum flow.

Power.—Excellent reservoir sites have been found on the head-waters and along the main channels of the various tributaries, by utilizing which a considerable portion of the flow could be stored for power and irrigation. Such storage would equalize the discharge, that for power having the greater relative influence. There are at present no water-power plants of any importance whatever in the whole drainage area of the Green River. A total of approximately 40 000 h.p. has been developed in the Grand, 7 000 in the San Juan, and 20 000 in the Gila Basin, in connection with irrigation construction. No data seem to be available as to the amount of energy which it is commercially practicable to develop under existing conditions on these various streams—it is obvious that there must be a vast difference between the figures for theoretically possible and for commercially feasible developments.

Irrigation.—The water of the streams making up the Colorado is already utilized for irrigation to a considerable extent. The oldest and largest development in the basin is perhaps that on the upper Green River, in Wyoming. Recently, large irrigation systems have been constructed in the Duchesne River Basin, and there is considerable irrigation around Vernal, and also Green River, Utah. Along the White and Yampa Rivers, in Colorado, meadow irrigation is extensively practiced, and projects are on foot for the irrigation of from 200 000 to 300 000 acres in this section.

Similarly, in the Grand Basin, there are extensive meadow lands in the upper part, and a half dozen small projects in contemplation for the Middle Basin which together would irrigate about 35 000 acres. In the Lower Basin is the Grand Valley Project, covering an irrigable area of 70 000 acres, and the Uncompahgre Valley Project, which, when completed, will irrigate about 150 000 acres, both by the United States Reclamation Service. Under other schemes, from 40 000 to 50 000 acres more will be irrigated.

Quite a little land along the San Juan, Animas, Pine, Florida, and La Plata Rivers, and the small tributaries of the San Juan, in Colorado, is now under cultivation, and also several thousand acres of valley land in New Mexico, but, as yet, irrigation has largely been confined to the bottom lands. The greatest probability of future irrigation development in this basin is in San Juan County, New Mexico, where it is said that probably 1 000 000 acres of fertile lands are excellently adapted for irrigation, for which the water supply is ample, the average annual run-off at Turley dam site being probably more than 1 000 000 acre-ft., and the reservoir at that point having a capacity of 1 500 000 acre-ft.

In the Little Colorado River Basin there are scattered a few relatively unimportant patches of irrigated land, while the U. S. Reclamation Service has investigated and found feasible the irrigation of approximately 70 000 acres in the vicinity of Holbrook, by constructing storage reservoirs at St. John's and Woodruff, Ariz.

There are also irrigation possibilities in the Virgin River and Bill Williams Fork Basins, but their total area is relatively unimportant, as far as concerns their effect on floods, or the irrigation of lower lands.

There are excellent opportunities for irrigation in the Gila River Basin, chief of which are the projects examined by the U. S. Reclamation Service in the vicinity of Alma and Lordsburgh, N. Mex. At the latter point there are 250 000 acres of almost unbroken and very fertile land which could be irrigated by the stored water of the Gila River, although at considerable expense. Other good storage sites exist at San Carlos on the Gila, and at Roosevelt on the Salt, the latter having already been utilized by the U. S. Reclamation Service by building the famous Roosevelt Dam, behind which can be stored 1 100 000 acre-ft. of water. With this water, about 200 000 acres of land will be irrigated directly, and power will be generated for pumping water to nearly 60 000 acres more. In addition, there is an excellent reservoir site on the Verde River above McDowell, and large tracts of land on the Gila River in the vicinity of Solomonville and of Florence, Ariz., are now irrigated.

Along the Colorado River itself there are storage sites at Bullhead Point and at another point about 6 miles above the Laguna Dam near Yuma, while there are irrigable lands between Mohave and Yuma aggregating some 400 000 acres.

Table 1 is a summary of the areas above Yuma which are now irrigated, in a technical sense, although much of this territory, no doubt, is watered in a very unsatisfactory manner.

TABLE 1.

District.	Acres.	Acres.
Colorado River direct	19 000	
Green River and tributaries	255 000	
Grand River and tributaries	305 000	
Fremont River	16 000	
San Juan River and tributaries	57 000	
Little Colorado River and tributaries	12 000	
Virgin River	16 000	
Gila River and tributaries	230 000	
Scattering (other tributaries)	7 500	917 500
Additional Irrigable Lands above the Yuma Valley.		
Above the Grand Cañon	450 000	
Colorado River Valley below Mohave	400 000	
The Gila Drainage Basin	400 000	1 250 000
Irrigable Lands in the Delta.		
Yuma Project	90 000	
Imperial Valley in the United States	600 000	
Imperial Valley in Mexico	300 000	
Other lands in Mexico—east of the Colorado	200 000	1 190 000
Grand total	.....	3 357 500 acres.

TABLE 2.—Approximate Storage Possibilities of the Basin.

	Acre-feet.
Green River, including the Brown Park Reservoir site	3 000 000
Grand River, including the Kremmling Reservoir site	3 000 000
Little Colorado	50 000
Bill Williams Fork	100 000
San Juan	1 500 000
Virgin River	....
Gila River	2 500 000
Colorado, below Mohave and above Yuma	....
Total	10 150 000 +

It must be borne in mind that all the figures in Tables 1 and 2 are for developments which are theoretically possible, and they would have to be more or less seriously reduced to be correct for commercially

feasible developments, on account of the excessive cost and the formidable character of the silt problem.

Discharge at Yuma.—Observations of the gauge heights of the Colorado River have been made by the Southern Pacific Company on its bridge at Yuma since 1878. The U. S. Geological Survey has maintained a gauging station at this point since 1895, using rating curves for discharge reductions until 1902, since which time careful current-meter observations have been made every 3 or 4 days. Table 3 contains the data thus collected for the 18-year period, 1894 to 1911, reduced to averages.

TABLE 3.—ANNUAL DISCHARGE OF COLORADO RIVER FROM 1894 TO 1911, INCLUSIVE.

Year.	Mean, in cubic feet per second.	Total acre-feet.
1894	7 400	5 390 000
1895	9 900	7 162 000
1896	9 000	6 515 000
1897	12 400	9 039 000
1898	9 100	6 581 000
1899	12 200	8 870 000
1900	9 400	6 798 000
1901	11 700	8 495 000
1902	8 400	6 127 000
1903	15 600	11 323 000
1904	13 900	10 119 000
1905	27 300	19 710 000
1906	26 800	19 475 000
1907	35 100	25 500 000
1908	18 900	13 700 000
1909	35 800	26 000 000
1910	19 700	14 335 000
1911	24 600	17 839 000
Mean	17 070	12 388 000

The minimum annual discharge was observed in 1894, and the maximum in 1909. The discharge has been strikingly greater since 1902 than for previous years, but too much dependence should not be placed on the data obtained prior to 1902, at which time very frequent current-meter observations were commenced. The lowest discharge was probably 2 400 sec-ft. in January, 1894, the average for that month being only 2 510 sec-ft.; the greatest was 149 500 sec-ft. on June 24th, 1909. The smallest total discharge for one month was 154 100 acre-ft. in January, 1894, and the greatest was 6 250 000 acre-ft. in June, 1909.

TABLE 4.—MEAN MONTHLY DISCHARGE OF COLORADO RIVER, 1894 TO 1911, INCLUSIVE.

Month.	Cubic feet per second.	Total, mean monthly, in acre-feet.
January	7 340	450 400
February	8 370	466 900
March	12 830	787 800
April	16 380	973 200
May	34 280	2 104 200
June	50 500	3 000 000
July	29 630	1 819 200
August	13 560	832 700
September	9 880	586 900
October	8 460	519 000
November	6 660	395 900
December	7 060	433 200
Totals	17 080	12 369 400

The record for 1908 is given by months in Table 5 as typical of the monthly variation. The lesser disturbances caused by the floods from the Gila in the autumn are very well shown; in this case, the maximum discharge from this source occurs in December, instead of from the Colorado in June.

TABLE 5.—MONTHLY DISCHARGE OF COLORADO RIVER AT YUMA, ARIZONA, FOR 1908.

(Drainage area, 260 000 sq. miles.)

Month.	Discharge, in Second-Feet.				Run-Off.	
	Maximum.	Minimum.	Mean.	Per square mile.	Depth, in inches, on drainage area.	Total, in acre-feet.
January	7 400	5 600	6 320	0.028	0.03	389 000
February	45 000	6 300	14 200	0.063	0.07	817 000
March	33 000	10 100	16 100	0.072	0.08	990 000
April	35 000	12 900	17 800	0.079	0.09	1 060 000
May	33 000	23 000	27 200	0.121	0.14	1 670 000
June	61 700	30 000	42 900	0.191	0.21	2 550 000
July	53 800	18 900	32 600	0.145	0.17	2 000 000
August	36 100	18 600	24 300	0.107	0.12	1 490 000
September	19 300	7 000	11 400	0.051	0.06	678 000
October	20 600	6 600	9 510	0.042	0.05	585 000
November	10 200	6 000	8 090	0.036	0.04	481 000
December	72 500	6 000	15 900	0.071	0.08	978 000
The year	72 500	5 600	18 900	0.084	1.14	13 700 000

Necessity for Storage.—The figures for the discharge at Yuma show that, in an ordinary dry year, the Colorado, without regulation, will

serve not more than 500 000 acres. On the other hand, in an ordinary dry year, with fairly complete regulation—that is, with 2 000 000 acre-ft. of water storage—this river will serve 1 500 000 acres, and any supply held over from wet to dry years would add to the reserve. It is conservative to assume at present that no reservoir site on the Colorado below the Grand Cañon can be utilized, on account of the apparent absence of rock foundations for dams in the river, while, even if other things were favorable, the tremendous quantity of silt in the water means a heavy reduction in the reservoir capacity which could be obtained. Indeed, it has been seriously suggested that by the construction of a series of such dams, the silting up would in time create large areas of excellent land, one above the other.

Above the Grand Cañon, the Kremmling Reservoir site, on the Grand River, and the Brown Park Reservoir site, on the Green River, would together store approximately 4 500 000 acre-ft., and thereby add much more than 1 000 000 acres to the irrigable lands of the Arid West. When it is considered that the present irrigated area of Southern California, exclusive of the Imperial Valley, is less than 300 000 acres, the potentiality of storage along the Colorado is startling.

Another very important feature of water storage along the river is the marked effect it would have in decreasing the difficulty of controlling the Lower Colorado River. Levee construction and bank protection must obviously be designed to guard against maximum floods, and it is these which the storage basins would affect to the greatest degree. The completion of the Roosevelt Dam, which will hold back 1 100 000 acre-ft. on the Salt River, will in future undoubtedly reduce the dreaded floods from the Gila River.

Rise of the Bed at Yuma.—If the measured discharge of the river at various heights is used in making a rating curve, and this curve is extended back, by means of the gauge readings, to 1878, the results would indicate that the quantity of water formerly passing Yuma was materially less than at present. As a matter of fact, the average low-water plane has constantly risen, and a comparison of the gauge heights by 10-year periods beginning with 1878 shows the following average elevations:

1878 to 1889	114.5 ft.
1890 " 1899	116.6 "
1900 " 1909	117.4 "

The low-water plane at the end of 1909, however, was 3½ ft. lower than during any of the six preceding years, which included the period of diversion into the Salton Sea. Indeed, it was lower, by more than 1½ ft., than 20 years ago, and only 0.8 ft. higher than during 1878-79. The reasons for this interesting condition of affairs will be considered later.

Following conventional practice, the endeavor was made for a long time to establish a rating curve for the Yuma gauging station, but this was found to be impossible. The reason is that the bed is eroded during high water and silted up during lower stages, thus fundamentally changing the cross-section, not only for different gauge heights, but for the same gauge heights at the beginning and end of

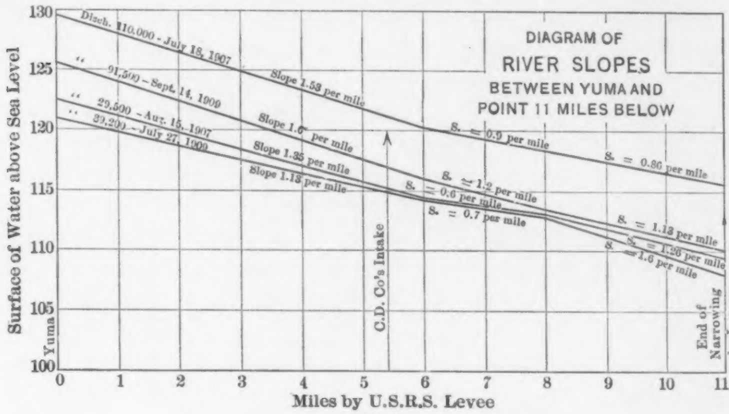

FIG. 2.

a high-water period. The reason for the exaggerated extent of such action is as follows: The Colorado at all times carries considerable silt, the quantity and character, of course, depending on the velocity of the water. Assuming a given discharge, and conditions of equilibrium, the bed of the river will have a given slope, the water will have a certain velocity, and will carry a certain quantity of sediment, none of which will exceed a definite size or specific gravity. If the volume of water increases, the water section and hydraulic radius will increase, and will result in greater velocity, which will give greater silt-carrying capacity. Conditions at the outfall or mouth are determined and temporarily unchangeable, therefore, it follows that the grade of the river will automatically tend to flatten itself by

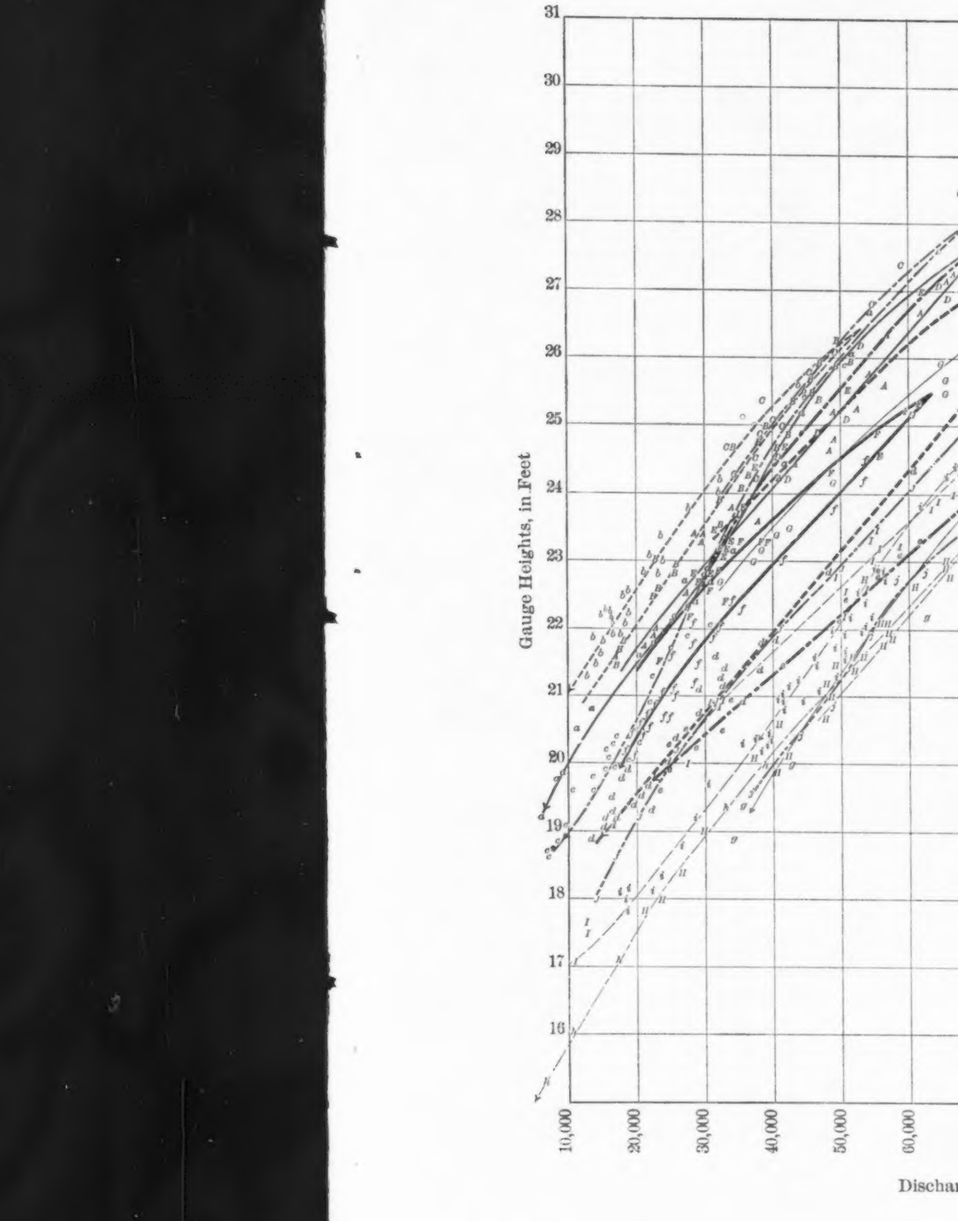
Gauge Heights, in Feet
31
30
29
28
27
26
25
24
23
22
21
20
19
18
17
16
10,000
20,000
30,000
40,000
50,000
60,000
Dischar

PLATE CI.
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CORY ON
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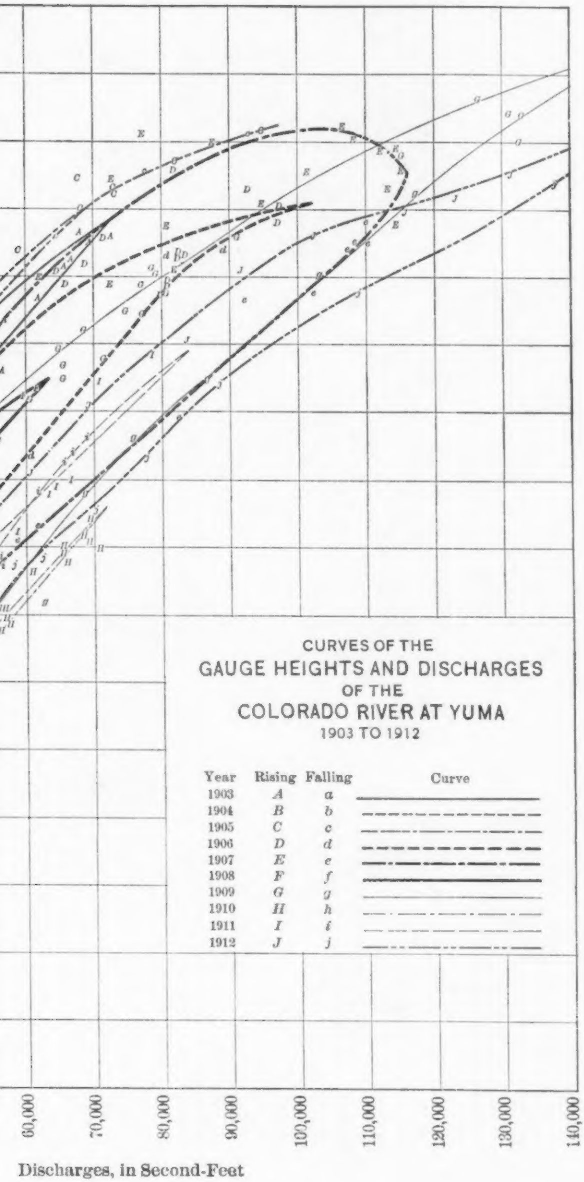
COLORADO RIVER DELTA.



picking up additional quantities of silt and carrying them along. When the volume of water decreases, the velocity will slacken, resulting in carrying less silt, and the bottom will rise with increasing slope until equilibrium is again established. This condition of affairs results in surprisingly great changes at Yuma during long periods of high water. In 1907 and again in 1909 it was found that for an increase of 10 ft. in the gauge height there was a lowering of the bed of approximately 30 ft., making the total increase in depth of water almost 40 ft. In other words, the grade line drawn from the bottom of the channel at Yuma to the average water surface in the Gulf of California had 30 ft. more fall, from Yuma down, at the beginning of the summer floods of 1907 and 1909, than when the peaks had just been passed. A few weeks after the first of these floods had entirely passed, the bed of the river had been restored to its usual low-water position.

When flashy floods occur, there is not sufficient time for this action to take place to a marked degree, and therefore the flashy rise of November 28th, 1905, having an estimated discharge of only 115 000 sec-ft., reached a gauge height of 31.3 ft., whereas the maximum discharge in the summer flood of 1909 was 149 500 sec-ft. and the gauge height was only 29.2 ft. In other words, the flashy floods do not have time to render the river channel more efficient before the maximum demand is made on it.

The increase in the gauge height of the low-water plane is due to the same general action. As the river builds the delta farther and farther into the Gulf of California, the bed must rise all along the line, of course, taking averages of considerable periods of time. According to Capt. J. H. Mellon, of Yuma, Ariz., who for a great many years navigated the Lower Colorado, the delta fan has extended out into the Gulf more than 6 miles in the past 40 years. Assuming the fall of the river in the lower reaches at 1.2 ft. per mile, the rise in the bed should average 1.2 ft. in $6\frac{2}{3}$ years, or approximately 0.2 ft. per year. These figures are about what the hydrographs seem to show, namely, 2 ft. per 10-year period.

Effect of 1909 Flood.—The fact has been mentioned that the low-water plane at the end of 1909 was only 0.8 ft. higher than during 1879, and this becomes much more striking when the general elevations for the entire period are shown by a curve. There were two

factors which tended to produce such a result: first, the diversion of the river through the Abejas to the west during the summer flood of 1908, and the lowering of the river bed at that point; and second, the effect of the Laguna Weir basin, which existed as such for the first time that year.

It seems very probable that the Abejas diversion was the smaller influence, in spite of the fact that at the time it was generally considered to be the only factor of importance. Undoubtedly, the bed of the river, and consequently the surface of the water, lowered rapidly while the diversion was becoming an accomplished fact. The amount of such lowering could not have been more than a very few feet at most, although it probably seemed much greater to nervous and frightened observers.

Doubtless it was an important factor that the Laguna Weir had been completed just before the beginning of that year's summer flood, and created a reservoir having a capacity of perhaps 20 000 acre-ft. The waters of the Colorado, heavily laden with silt, were here stilled and their contents deposited. The large volume of water which passed over the dam—the greatest ever recorded on the river itself—contained little more silt than it would ordinarily during low-water stages. Consequently, it picked up and carried along the silt to an unprecedented extent. As the waters receded, the bed was built back to a very much less extent, because there was still an extraordinarily small quantity of silt in the water. Indeed, during this one season, the basin formed by the Laguna Weir was completely filled and some 20 000 acre-ft. of mud were deposited out of the Colorado at this point instead of being spread along the river bed thence to the Gulf.

Unfortunately, no sediment observations were made at Yuma during this flood period. Had this been done, the influence of the Laguna Basin on the low-water plane would doubtless have been approximately ascertainable. In any event, the gauge heights at Yuma, for discharges of 30 000 and 10 000 sec-ft., respectively, platted as ordinates, with the times as abscissas, as in Fig. 3,* for the period of 1902 to 1912, show very clearly that there has been no serious grade recession at Yuma due to the Abejas diversion.

* This method of platting seems to be the only one possible to show much relation, if any at all, between gauge height, discharge, and time at the Yuma gauging station.

GAUGE HEIGHTS OF COLORADO RIVER AT YUMA
FOR DISCHARGES OF 30000 AND 10000 CU. FT. PER SEC.
JANUARY, 1903, TO JUNE, 1912.

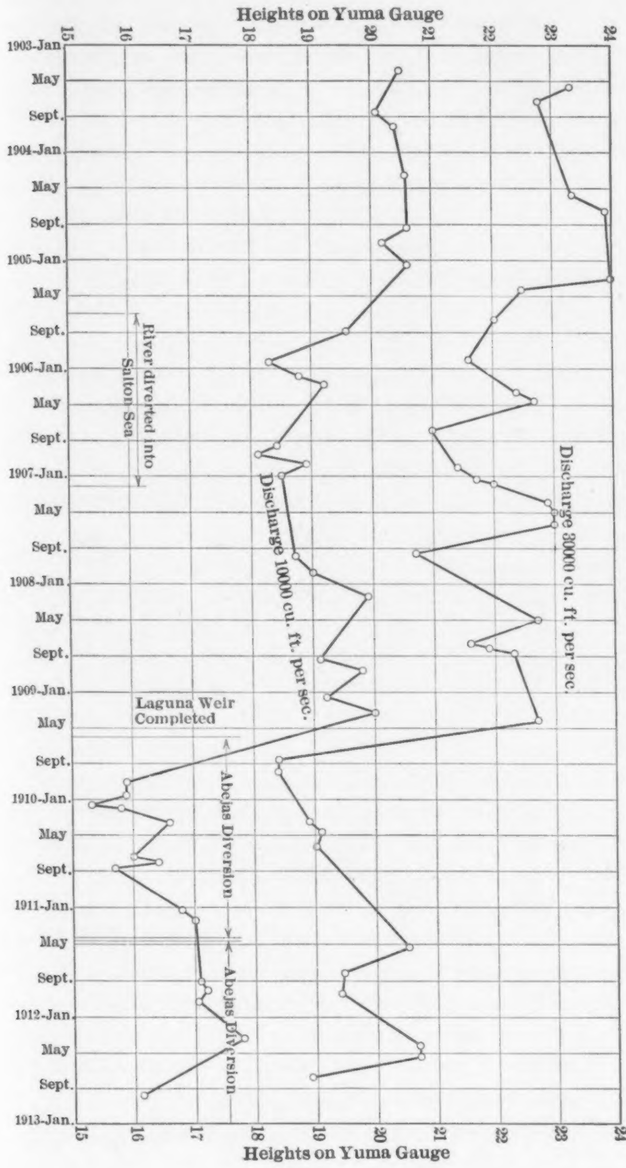


Fig. 3.

Silt.—Professor C. B. Collingwood, of the University of Arizona Agricultural Experiment Station, examined the silt contents of the Colorado River water at Yuma for a period of 7 months, beginning with August, 1892. One pint of water was taken each day and evaporated, and the sum of the daily residues for each month was then weighed and analyzed. The results varied from a minimum of 1 part of sediment to 613 parts of water in January, 1893, to a maximum of 1 to 97 in October, 1902, and an average of 1 to 388, the ratio of dry material to weight of water being 1 to 277. The corresponding ratio in the Mississippi is 1 to 1 500; the Nile, 1 to 1 900; the Danube, 1 to 3 060. The average value of the fertilizing material was computed at $3.22 per acre-ft.*

Later, January 1st to December 31st, 1904, Professor R. H. Forbes, now Director of the same Experiment Station, made a careful study of the quantity of silt,† and the relation of irrigating sediments to field crops.‡ It was found that the quantity of silt varied from 84 to 3 263 parts per 100 000 by weight, and from 250 to 9 800 parts per 100 000 by volume, or roughly $\frac{1}{1\,200}$ to $\frac{1}{30}$ part by weight, and that 1 acre-ft. of river water contained from a minimum of 1.14 tons to a maximum of 44.42 tons, and an average of 9.62 tons, of silt. Obviously, the total quantity of sedimental material cannot be obtained by multiplying the average sedimental contents by the total annual discharge, but the investigations were carried out in such detail that it was possible to compute the quantity of solid material from the discharge at the time, and in this way it was found that the total solid material carried past Yuma that year was 120 961 000 tons. The total discharge of the river during that year was 10 119 000 acre-ft., while the annual average for 1894 to 1911, inclusive, was 12 388 000 acre-ft. It would seem conservative to estimate that the average quantity of material would be as much larger than that delivered in 1904 as the discharge, on which basis the result would be $120\,961\,000 \div 10\,119\,000 \times 12\,388\,000 = 148\,084\,000$ tons. The specific gravity of the Colorado sediment is 2.65 and the weight of dry soil is 93 lb. per cu. ft., so that this quantity of material would make approximately 71 800 acre-ft. or 112 sq. mile-ft. of equivalent dry alluvial soil.

* These results are given in Bulletin No. 6 of the Arizona Agricultural Experiment Station.
† Bulletin No. 44.
‡ Bulletin No. 53.

Navigability.—In a technical sense, the Colorado River is navigable from its mouth up to Laguna Dam, and again from there to The Needles. This navigability was recognized when Mexico and the United States entered into the treaty of 1848 regarding the International Boundary Line. By the provisions of this treaty, neither country was to permit works which would interfere with navigation throughout that part of the river which is a common boundary. In a subsequent treaty (1853) this provision was abrogated, but the United States guaranteed in lieu thereof a free and uninterrupted passage of vessels and citizens as far as the river forms a common boundary. As a matter of fact, the swift, shoal waters and the shallow depth over bars in the river itself, together with a tidal bore at the mouth, where the range of tide exceeds 30 ft., has resulted in practically no commerce on the river below Yuma since the Southern Pacific Railroad completed its track in 1876. At various times the U. S. Army engineers have investigated the situation, but have always reported that the navigation interests were not sufficient to justify any expenditure for river improvement.

An Act approved April 21st, 1904, authorized the Secretary of the Interior to divert water from the Lower Colorado River for irrigation purposes and to construct a diverting weir across the river at The Potholes, or Laguna, in which no provision whatever is made for navigation.

Delta of the Colorado.

The Delta of the Colorado River of the West, at the head of the Gulf of California, lies approximately between the parallels of 32° and 33° N. and the meridians 114° 30′ and 115° W. It is partly north of the International Boundary Line between the United States and Mexico, and in larger part south of that line. Its area, including the Pattie Basin and the Cocopah Mountains, is approximately 6 000 sq. miles. It extends practically from the mouth of the Gila River, at Yuma, westward to the rocky walls of the San Jacinto Mountains and south to tide water of the Gulf, while on the north it blends with the depressed area below the sea-level, from which the ocean has been cut off by the deposits of the stream. Its general deltoid form is shown on Fig. 4, together with the course of the main stream and principal branches, sloughs, and overflow channels.

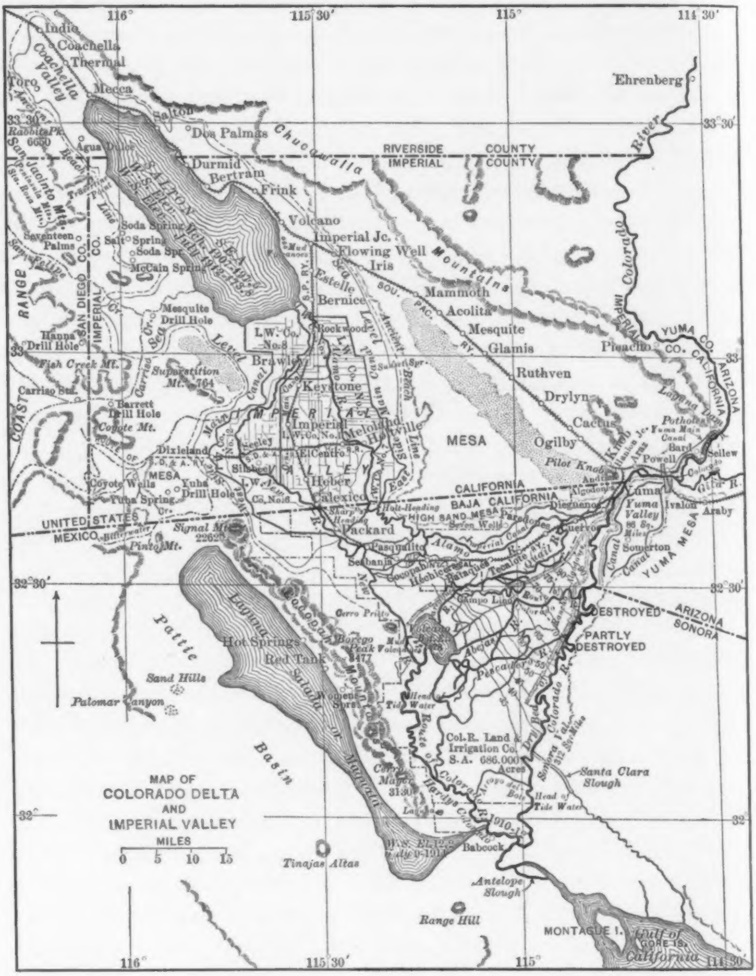

MAP OF
COLORADO DELTA
AND
IMPERIAL VALLEY
MILES
0 5 10 15
116°
115° 30′
115°
114° 30′
33° 30′
33°
32° 30′
32°
Indio
Coachella
Thermal
Mecca
Coachella Valley
Salton
Dos Palmas
Chuckawalla Mountains
RIVERSIDE COUNTY
IMPERIAL COUNTY
Ehrenberg
Colorado River
Durmid
Bertram
Frink
Volcano
Imperial Jc.
Flowing Well
Iris
Mammoth
Acolita
Mesquite
Glamis
Ruthven
Drylyn
Cactus
Ogilby
Pilot Knob
Picacho
YUMA CO.
CALIFORNIA
ARIZONA
Laguna Dam
Potholes
Bard
Powell
Sellew
Gila R.
Yuma
Yuma Valley
Ivalon
Araby
Somerton
YUMA MESA
Estelle
Bernice
Rockwood
Brawley
Keystone
Imperial
Meloland
Holtville
El Centro
Seeley
Silsbee
Dixieland
Heber
Calexico
MESA
Coyote Wells
Yuha Drill Hole
Yuha Spring
Mesquite Drill Hole
Hanna Drill Hole
Fish Creek Mt.
Superstition Mt.
Carriso Sta.
Barrett Drill Hole
Coyote Mt.
Soda Spring
Salt Spring
McCain Spring
Seventeen Palms
Rabbit Pk. 6650
Agua Dulce
Toro
SAN DIEGO CO.
IMPERIAL CO.
COAST RANGE
UNITED STATES
MEXICO
Signal Mt. 2262
Pinto Mt.
CALIFORNIA
BAJA CALIFORNIA
HIGH SAND MESA
Packard
Pasqualito
Sesbania
Alamo R.
Cocopah Mts.
Cerro Prieto
Borrego Peak 3477
Hot Springs
Red Tank
Laguna Salada
Pattie Basin
Sand Hills
Palomar Canyon
Women Sprs.
Campo Lind
Abejas R.
Pescadero R.
Colorado R.
DESTROYED
PARTLY DESTROYED
ARIZONA
SONORA
Col. R. Land & Irrigation Co. S. A. 686,000 Acres
Santa Clara Slough
Head of Tide Water
Hardy Colorado
Babcock
Antelope Slough
Tinajas Altas
Range Hill
MONTAGUE I.
GORE IS.
Gulf of California

FIG. 4.

The Lower Colorado River.—The Lower Colorado River may be considered as that portion lying below the last narrows, at what is known as The Potholes—the location of the Laguna Dam, of the United States Reclamation Service. At this point the river debouches

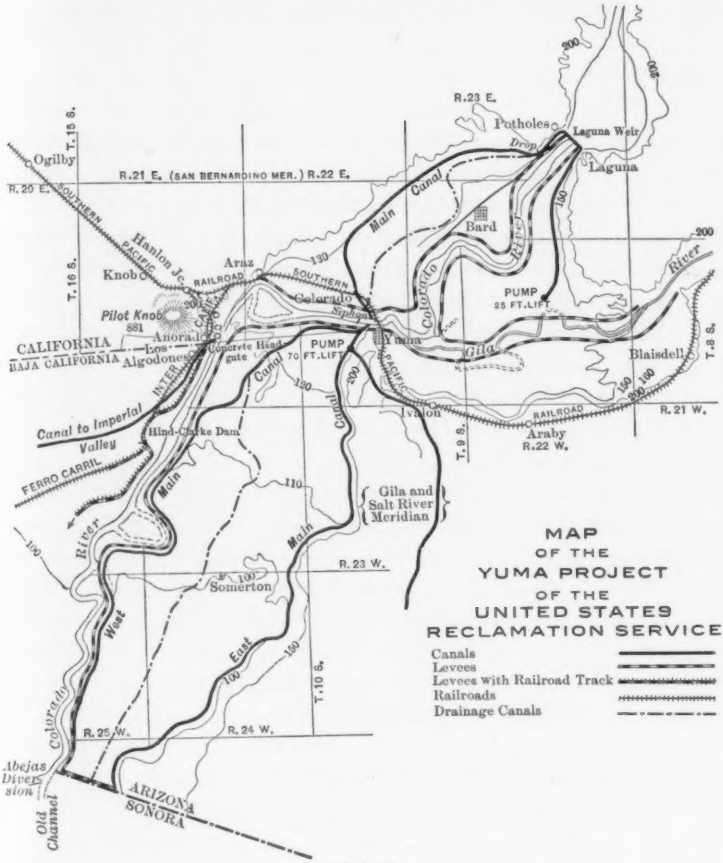


FIG. 5.

upon the plain, and the valley on each side is bounded by diverging mesas. About 13 miles below, and just above Yuma, the Gila River joins it. The present location of this portion of the river is shown on Fig. 5.

Alignment.—Below The Potholes there are two controlling points: one is a peculiar knob of indurated clay at Yuma through the center

of which the river channel has passed since the first advent of the whites; the other is the granite hill known as Pilot Knob. The small eminence at Yuma covers about 40 acres, and reaches a height of not more than 100 ft. above the general level of the delta plain. A similar though much smaller knob lies on the east bank of the river, about 1 000 yd. below and just to the south of the Southern Pacific Railroad Company's line and bridge, and is occupied by the reservoir and settling basins of that company. These peculiar topographical features control the river, with respect to its location at Yuma, and at Andrade, at the International Boundary Line, 8 miles farther down.

Grade.—The course of the river is quite winding, like every flashy, silt-bearing stream with a relatively steep grade. The elevation at The Potholes is approximately 140 ft. above sea level, and the distance by the river is about 100 miles to the head of tide-water and 114 miles to the mouth at the Gulf. Thus the general average fall is approximately 1.3 ft. per mile.

Remarkable Vegetation.—Attention must be called to the dense and varied vegetation throughout the region subject to the river's overflow. Arrow weed grows in nearly impenetrable jungles; mesquite and screw-bean trees occur in forests of varying density on older established soil, while freshly deposited mud flats and banks are almost immediately covered with seedling willows which quickly grow into heavy timber. For instance, Professor Forbes counted on an area 5 ft. square 1 500 willow sprouts up to 20 in. high, and in another older growth 90 young willow trees 20 ft. high.* Cottonwoods occur, but are not abundant. Along the river banks and sloughs there are dense thickets of common wild cane, which the Mexicans call *carrizo,* with a densely matted root stock which affords great resistance to erosion of the soil because the plant spreads both by means of these root stocks and by sending long slender stems or runners across the mud flats to distances of 20 or even 30 ft., and these strike root at every point, thus rapidly establishing the plant on newly made ground. In marshy locations are found great fields of a plant with an immense edible bulb used by the Cocopah Indians as a food, locally known as *tule.* In addition there is the *sesbania,* or so-called wild hemp, which is limited strictly to ground subject to overflow. It comes up from seed

* "The Lower Courses of the Colorado," R. H. Forbes, in *The Great Southwest*, Yuma, Ariz., Vol. 1, Oct., 1906, p. 2.

annually after the subsidence of the summer floods, stands in dense thickets from 5 to 20 ft. high, and is often square miles in area. This plant is also of interest because of its industrial possibilities. In general, the vegetation of the delta is remarkable for the manner in which plants of a kind mass together in areas almost to the exclusion of other species, and for the remarkable density and immense areas occurring in continuous bodies, strips, and patches, particularly of willow, arrow weed, wild hemp, and *carrizo*.

Line Changes.—The entire Colorado River Delta has been said to consist of alluvial silt. When the river is low the water wanders in a devious way, along a very wide shallow bed in many places, and is everywhere confined by banks seldom exceeding 10 or 12 ft. high. During high stages these banks are overflowed at many points, and in the case of severe floods such overflow is practically general. The banks are thus wet and softened, and, when the river falls, caving and side-cutting proceed wherever the current is thrown at an angle against the confining banks, and often with startling rapidity. At the same time, the overflow water, being very heavily charged with silt, is checked by the dense, matted growth, and at once deposits its heavier particles, the smaller sizes being dropped a little farther down stream, and so on. Thus the country is built up most rapidly at the banks, and the land slopes away from the river at a constantly decreasing rate. Indeed, the theoretical cross-section of the land surface away from the stream is a hyperbola. Of course, these slopes are not identical at any two points along the river, but instrumental data at present available show the general average fall to be about 1½ ft. in the first 100 ft.; 3 ft. in the first 300 ft., and from 5 to 8 ft. in the first 3 000 ft.

Although the coarser silt deposits are thus found immediately at the river bank, there are several reasons why this has little practical significance. The overflow water gathers in little channels which follow the line of greatest slope and in general approximately away from and down stream, the direction being the resultant of the general grade parallel to the river, and of the slope locally from the river's banks to the abeyment on either side. Such overflow channels build up their miniature beds and banks exactly like the main channel; they join to form overflow creeks, and these in turn form the overflow rivers.

As the level of the river rises higher and higher by such overbank deposition, it is obviously only a question of time until an unusual flood will produce sufficiently high velocities in some of these overflow channels to cause a recession of their grades extending through the river bank, thus diverting a portion of the water through the new route. Ordinarily, as the flood recedes, such breaks are clogged with drift and sediment, but sometimes the clogging action is not rapid enough to counteract the opposing forces successfully, and in this way radical and extensive changes of the river's course throughout the delta occur. Usually, these changes are in the nature of cutting off bends and thus shortening the channel.

At first thought it would seem that a diversion to the west would be a very probable occurrence during any great flood, because, with the constant extension of the delta southward, the gradient in that direction has become less, and to the west, more, until the fall toward the Gulf is much less than half as great per mile as that along former courses to the Salton Basin. As a matter of fact, however, though the overflow waters go down these channels with considerable rapidity, the cross-sections for many miles from the river are exceedingly inefficient, due to the dense vegetation, drift in the water, and occasionally, no doubt, to beaver dams.

In addition to the foregoing, there is another factor of importance: The bed of the main stream for quite a distance on each side of the International Boundary Line is excessively eroded during flood periods and filled up during lower water stages, as has been fully explained, so that, with a given flood discharge in the river, the water going over bank constantly decreases in quantity, in depth, and in velocity, and it is only the overbank flow which is important in connection with the overflow channels.

Character of Local Silt Deposits.—These various actions result in the formation of numerous little pockets throughout the inundated areas, in which water is left standing after the recession of each flood. Wherever this occurs the very finest of the silt settles out and, on becoming dry, cracks in large, somewhat hexagonal, irregular cakes. If the deposit is very thin, these cakes curl up when thoroughly dried and are broken up and carried away as dust by the wind; but when very thick the cracks are sometimes 6 in. and even more in width at the top and extend down 4 or 5 ft. Dust and vegetation accumulate in

PLATE CII.
PAPERS, AM. SOC. C. E.
NOVEMBER, 1912.
CORY ON
IRRIGATION AND RIVER CONTROL,
COLORADO RIVER DELTA.

FIG. 1.—TYPICAL SURFACE OF CRACKED ADOBE SOIL.

FIG. 2.—TYPICAL SUBSURFACE OF CRACKED ADOBE MATERIAL. THE BOTTOM OF THE ROD IS 5.3 FEET BELOW THE SURFACE.

such gaping cracks, and the next flood deposits another layer of sediment. Then, the heavier materials having settled first, the result is that the pockets are arched over, thus producing underground interstices which must be carefully guarded against in levee and other earth construction for holding back water.

The character of such deposits depends on the nature of the silt carried in the overflow water, and thus it happens that it is usually possible by examination to determine whether a deposit was made by a flood from the Colorado, or from the Gila proper, or from the Salt River.

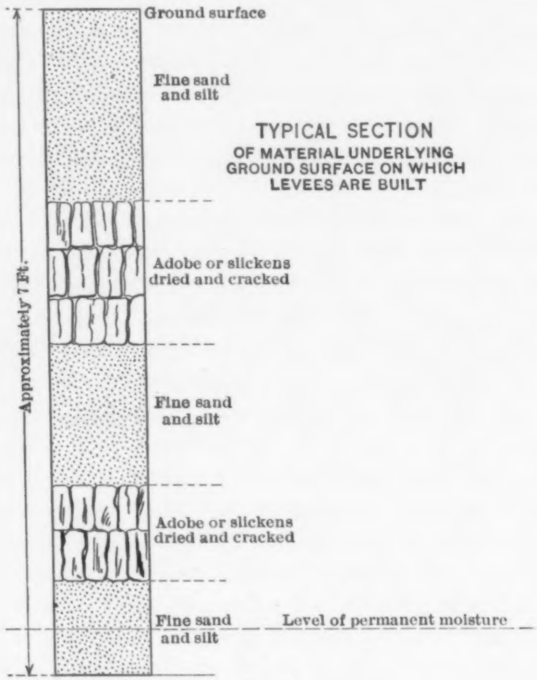


FIG. 6.

The rate of such local deposition is sometimes startling. One instance the writer observed was due to the flood of 1905 which caused the diversion of the river into the Salton Sea. This filled in the ground on the left-hand side of the break about 3 000 ft. from the old river bank over quite an area to a depth of 6 ft., or almost to the roof of an Indian's *ramada*. In this manner the Colorado River.

from its exit from the Grand Cañon near The Needles, Cal., to about 3 miles south of Yuma, Ariz., has wandered about between its eastern and western abeyments, and, where these were any distance apart, has built up alluvial valley stretches which are practically level transversely.

The Principal Overflow Channels.—The overbank flow of the Colorado on the east side does not gather into channels of importance because the eastern abeyment is near by, except far down the stream, where there is what is known as the Santa Clara Slough. This, doubtless, at one time was the river's main channel, and during the severe summer flood of 1907 it carried so large a volume of water that for a time it seemed probable it would again become the main outlet of the river to the Gulf. This slough is about 40 miles long, and empties into the Gulf about 20 miles southeastward from the present mouth. This and the other smaller high-water channels on the east side are of no material importance in the engineering operations along the river.

By far the greater portion of the delta cone lies on the west side, where there are five inundation channels of considerable importance. These are shown on Fig. 4. In their order down stream from Andrade, they are known as the Alamo, New Paredones, Abejas, and Pescadero Rivers. Without a thorough knowledge of them and their relationship to the Colorado flood-waters, no satisfactory understanding of the problems of the Lower Colorado, and the endeavors to handle them, is possible.

The Alamo River, which was formerly often called the Salton or Carter's River, has its gathering ground in the northerly edge of the delta cone immediately south of Andrade. It follows somewhat closely the southern end of the sand hills, at times in a well-defined channel and again spreading out in broad swamp sections, known locally as lagunas. About 40 miles west of the Colorado it crosses the International Boundary Line, and occasionally its waters were doubtless carried clear into the Salton Sink. The swamp areas, *Las Lagunas,* were also drained in part by the Quail River, which emptied into the Paredones. Farther down the Alamo there is a low area to the south and west through which the overflow waters from the great flood of February, 1891, broke over from the Alamo into the New River, the main point being at what is known as Beltran's Slough. The latter runs into the low region between the Paredones and the Alamo, and

this drains into New River through the Garza Slough. It seems probable that in 1891, for the first time in many years, the overflow waters reached the New River channel directly *via* the Alamo, rather than *via* Volcano Lake. During this flood from the Gila and the later annual summer flood of the Colorado, sufficient water reached the Salton Sea *via* the Alamo and the New Rivers to cover approximately 100 000 acres in the bottom of Salton Sink to a depth of about 6 ft., and it is estimated that the discharge of both these rivers aggregated 17 000 sec-ft. for a period of several weeks. Well-defined channels in the soft alluvial soil were cut out by both these streams, and since then the New River has carried some water every flood season, as it did occasionally before.* In July, 1903, it reached a maximum of only about 4 000 sec-ft., which was then the largest since 1891.

New River really heads in Volcano Lake, and probably is what remains of an overflow channel through which the ancient inland lake, Lake Cahuilla, emptied into the Gulf of California. At present its grade is to the north and into the Salton Basin, and from the lake's edge it follows for some miles the base of the Cocopah Mountains until it reaches about the + 10-ft. contour, where the mountains turn rather sharply to the west. The river continues in a general northwesterly direction and crosses the International Boundary Line at Calexico, where, until the recent tremendous erosion due to the diversion of the Colorado River into the Salton Sea, it followed a gentle depression down the lowest median line into the Salton Sink. At a few places in its course it spread out into broad channels, a few feet in depth, and formed occasional ponds or lakes, the most important of which were the Cameron Lake, near Calexico, Blue Lake, a few miles farther northwest, and Pelican Lake, a few miles farther on. It is now a great barranca, averaging from 40 to 80 ft. in depth and 1 000 ft. in width, from a point about 6 miles southeast of Calexico to the Salton Sea.

The Paredones is the first drainage channel on the southerly slope of the delta cone, and within quite recent years had direct connection with the Colorado River. This connection was automatically reduced

* Old settlers in the vicinity agree in saying that in 1840, 1849, 1852, 1859, 1862, and 1867 large quantities of water reached the Salton Sea. In 1862 that body of water attained unusual size, and the flow in New River that summer was so great that it stopped the mail stage-line service between Yuma and San Diego for several weeks, and a flatboat was built to ferry across it.

to the very small channel which existed in 1906, when the levee construction then done fundamentally changed overflow conditions. The Paredones gathers the overflow water from a large area, and a few miles from the river becomes a channel of considerable width and depth, following thence along down an element of the delta cone. During the extraordinary conditions existing in 1905-06, it carried a very large quantity of drift, which, with the assistance of some beaver dams, accumulated about 7 miles above Volcano Lake, and resulted in enlarging the branches leading toward the south. The overflow water of this river gathers to the south in the Pescadero, and to the north joins with the similar water from the Alamo and runs in part to Volcano Lake and in part through Garza Slough to New River.

The Abejas River drains the overflow from the region immediately south of the Paredones, and empties into the western side of Volcano Lake. Since the summer flood of 1908, this channel has been carrying the entire low-water flow of the Colorado and the greater portion of the flood flow, which is the condition to-day. The reasons for this diversion and the efforts to stop it will be considered at length later.

The Pescadero, another important overflow channel, drains the region immediately below that unwatered by the Abejas. It empties into a network of channels which conduct the water from that part of the delta cone and including Volcano Lake, finally gathering into Hardy's Colorado and emptying into the Gulf.

Volcano Lake may be another remnant of the waterway through which the ancient Salton Sea drained to the Gulf. It is a flat basin, the bottom of which is about 22 ft. above sea level, and its high-water stage is about 35 ft. At such a stage it extends about 10 miles northwest and southeast, and is about 6 miles wide. It is fed by the Paredones and Abejas Rivers, the latter since 1908 being the course of the Colorado proper, and by the system of sloughs which form the Pescadero network and also serve as an outlet. It is on the summit of the low, flat divide between the Salton Basin on the north and the Gulf on the south, and thus its discharge is both toward the north and the south. From the size of the outlet channels it is obvious that the greatest discharge has in recent times been southward. Since 1908 a line of levees has prevented any water from passing into the New River and thence into Salton Sea; the lake's waters, therefore, go to

the Gulf through Hardy's Colorado, which is an important channel, averaging perhaps 500 ft. wide and 20 ft. deep at maximum stages, with a fall varying with the stage in the lake from less than 15 to more than 30 ft. in a distance of from 45 to 50 miles.

The engineering operations which resulted in the irrigation of the Imperial Valley and its threatened destruction by inundation at various times since, have in very large measure been concerned with the overflow channels just described.

Diversion to the West.—Regardless of the tendencies for and against a fundamental diversion toward the west, the Colorado continued to flow in its regular bed to the Gulf until 1905. There can be no doubt that the operations of the California Development Company, and particularly in making an artificial cut from the Colorado River into the Alamo Channel and the utilization of that channel as a main canal, rendered the diversion to the west at that point, when it broke through in 1905, very much easier and more probable of immediate occurrence. Nevertheless, the behavior of the river since that time indicates pretty clearly that a diversion to the west somewhere within the first 25 miles below Pilot Knob was just about due, under natural forces alone. The conditions of equilibrium had become unstable to a degree, and this is the condition in which they are to-day.

Mesa and Delta.—The high mesa land which forms the eastern abeyment below Yuma extends therefrom almost south and into Mexico. The river turns, crossing the valley almost from east to west for about 5 miles, until it reaches the foot-hills forming the west abeyment; then it turns more than a right angle, hugging these hills, to the International Line; and thence it flows for 80 miles, in a remarkably direct general line, but little west of south, to the Gulf. On the west side of the valley the foot-hills end at Pilot Knob, a small mountain at the International Boundary Line, and the low mesa begins. The edge of the latter runs southwest for 4 miles; then it turns sharply directly west for 25 miles; then again it turns sharply to a little west of north for 50 miles—the latter edge forming the east side of the cut-off portion of the Gulf, Lake Cahuilla.

It is thus in a sense almost proper to say that the Colorado Delta begins practically at the International Boundary Line between California and Lower California, and that, for the first 14 miles below that line, the river is running on the very edge of the divide of the delta

cones, on one side sloping northwest to the Salton Sea and on the other to the Gulf. Furthermore, from that point the river (until 1908) was in a ridge of its own making, which it was raising constantly, and which is quite close to the eastern abeyment.

Pilot Knob.—Pilot Knob is a small, detached, and relatively abrupt mountain lying just above the International Boundary Line on the west bank of the river, and is one of the landmarks of the region. One of its rocky arms extends almost to the present west bank of the river. Fifty years ago the river had a pronounced bend, shown by the dotted line on the map, Fig. 5, and hugged this rocky point until passing it. The time when the shift of the river took place is not definitely known, but, very fortunately, at present the alignment here for several miles is almost straight.

It is quite significant that Pilot Knob is the lowest point along the river where a canal can be taken out for the diversion of water, with the diverting structure resting on solid rock. For this reason, it has been considered as a strategic factor in the irrigation of the Imperial Valley, but, in the writer's opinion, quite erroneously. The engineering fetish of a solid rock foundation for structures for irrigation and other purposes confining water, has resulted in needlessly spending amounts of money in the United States alone which must aggregate a tremendous sum. Perhaps no case is more spectacular than that of Pilot Knob and its relation to the irrigation system of Imperial Valley.

Early Suggestions Regarding Salton Sea.—Almost the very first explorers were interested in the Salton Basin and its various possibilities. The ability to create an inland sea by diverting into it the water of the Colorado attracted much attention, and it was very seriously suggested because of a supposed advantageous effect that it might have on the climate of the entire region. On the other hand, the possibilities of irrigating the Colorado Desert by the waters of the Colorado, which has since been accomplished, were not overlooked, work having been done on many more or less serious propositions at various times.

Later Irrigation Projects.

In 1891 and 1892, the Colorado River Irrigation Company was formed. Mr. C. R. Rockwood was placed in charge of the engineering work, and, under his direction, the entire problem of irrigating the Colorado River Delta was carefully examined and the important

features fully worked out. The financial stringency of 1893 put an end to the operations of this corporation, and in 1894 Mr. Rockwood, was forced to sue the company for his unpaid salary. In partial satisfaction of the judgment which he obtained, the engineering records and data were taken over by Mr. Rockwood, and the Colorado River Irrigation Company ceased to exist. Nevertheless, it is interesting to consider the plans then evolved by that corporation, or, more properly speaking, by its engineer, Mr. Rockwood.

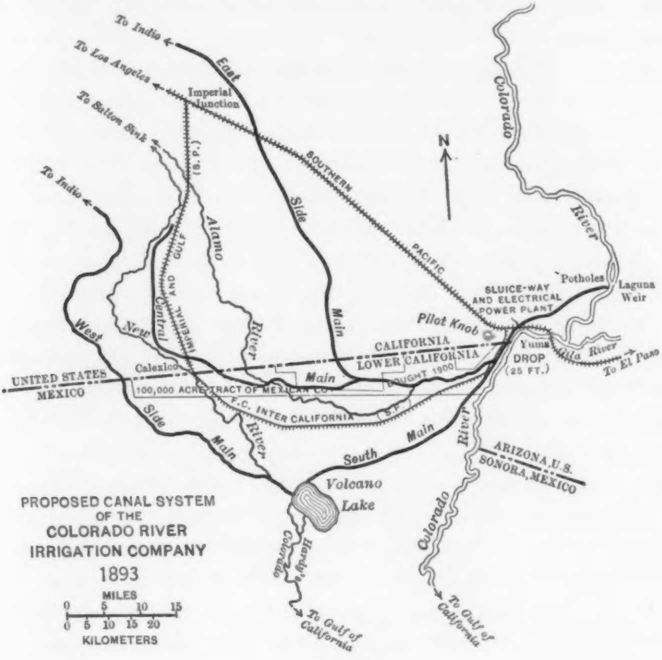


FIG. 7.

Plans of the Colorado River Irrigation Company.—These plans are outlined diagrammatically in Fig. 7, and show what is probably the ideal system of diversion and canals for watering all the land irrigable by gravity with the waters of the Lower Colorado. Events, however, shaped themselves so that the water for Imperial Valley has been, and is now being, diverted at Pilot Knob; while the U. S. Reclamation Service has built a diversion weir at The Potholes or Laguna to put water by gravity on all except the mesa lands in the so-called

Yuma Valley. Mr. Rockwood contemplated taking water out at The Potholes and installing in the main canal near Pilot Knob a sluice-way with which he intended to flush out the silt in the canal above, which escaped being deposited in and removed by hydraulic dredges from a short enlarged section of the main canal immediately below the diversion point, where the velocity of the water would be reduced. The dredges were to be operated by electricity generated at the sluice-way. The maps showing the detailed surveys of these canals are now in the files of the California Development Company at its Calexico headquarters.

The California Development Company.—Mr. Rockwood, being thoroughly imbued with the practicability and advantages of the project to irrigate the Imperial Valley with the waters of the Colorado, undertook to carry it through, and finally did so by means of the California Development Company. At the present time it is only important to say that, because of financial considerations, the engineering features were radically modified to diverting the water at Pilot Knob and utilizing a large part of the Alamo overflow channel as a main canal to carry the water around to the Imperial Valley, essentially as suggested by Lieut. Bergland in 1875-76. In this way the diversion work at The Potholes was eliminated, and a very cheap and quick method of getting water into the valley was arranged. By this decision the inclusion of the Yuma Valley as a part of the project was abandoned.

The Yuma Project, U. S. Reclamation Service.—As early as 1895 the Hydrographic Branch of the United States Geological Survey began stream gauging in California, starting with an allotment of \$5 000. More recently, the California Legislature has aided in the work on the basis of appropriating sums equal to those set apart by the United States. At the present time daily discharge observations are made on about fifty typical streams. Hydrographic investigations throughout the Western States, not only helped to prepare the way for national irrigation, but resulted in acquiring such hydrographic data that when the Reclamation Act was passed, in 1902, the best opportunities for national irrigation projects were pretty generally outlined. On account of legal and social complications elsewhere throughout California, the Yuma Project was finally selected as the first to be commenced by the Reclamation Service in that State. On April 8th, 1904, a board of seven engineers recommended this project; on April

21st, Congress authorized the Reclamation Service to take water from the Colorado and divert it by a weir which would close it to navigation permanently above Yuma. On May 10th, 1904, the Secretary of the Interior gave his approval, and an allotment of $3 000 000 was made.

There are approximately 75 000 acres of irrigable land under this project in Arizona, and 15 000 in California. Of this area, 98% is subject to the provisions of the Reclamation Act, the owners of private lands having signed the necessary agreements to limit their holdings to the size of the farm unit to be determined, and otherwise to conform to the regulations required by the Service. A Water Users' Association, consisting of the land owners of the project, handles the affairs of the district, from the farmers' point of view, and has contracted with the Secretary of the Interior to accept and use the water under the usual conditions fixed in such cases by the Government.

The Imperial Valley should logically have been included as a part of this project, particularly from an engineering point of view. However, water had been delivered into the Imperial Valley for almost 3 years when the Secretary of the Interior approved of the Yuma Project. In addition, there were complications—largely over-estimated and far more apparent than real—due to the fact that it is practically imperative to go through Mexican territory with canals to serve the American Imperial Valley. The project, therefore, was limited, for the present at least, to the irrigation of the Yuma Valley.

Fig. 5 is a map of the restricted Yuma Project. As it occupies a position on the river above that of the California Development Company's constructions, and for that reason in many ways has had a very important influence on the whole irrigation of the Colorado delta proper and related engineering problems, its essential features will be briefly described first. These are a diversion weir, and the levee, canal, and drainage systems. The diversion is by an overflow weir of the type developed by British engineers in their irrigation work in India, and improved and used later on the Nile. It is of loose rock, rests on a bed of river silt, is almost a mile long, very wide, quite low, and is in general an exceptionally interesting and expensive construction.

The next most unusual and interesting feature is the necessity for about 74 miles of levees to protect, from the overflow waters of

the river, the greater part of the land to be irrigated. The canal system, with the exception of the siphon under the Colorado, is nothing out of the ordinary, and the same is true of the drainage system.

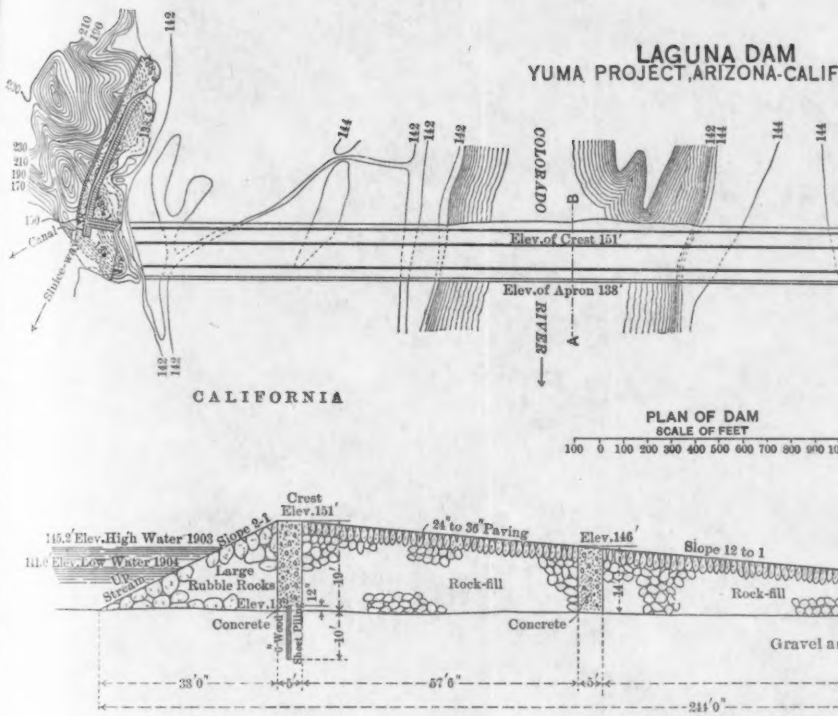
The project has proved very much more expensive than was originally contemplated, the estimated cost being $3 000 000, whereas, the construction expenditures up to June 30th, 1910, were $3 617 472.71,* exclusive of maintenance and operation charges and $100 000 of the preliminary survey costs more properly chargeable to general investigations along the Colorado River than to the Yuma Project itself. Work on the project was reported as 80.8% complete, but this estimate was revised in April, 1911, and changed from 81.6 to 52.4%, making the proper percentage completed on June 30th, 1910, about 51.8. On this basis, the total cost will be $6 964 233, or approximately $77.25 per acre of irrigable land.

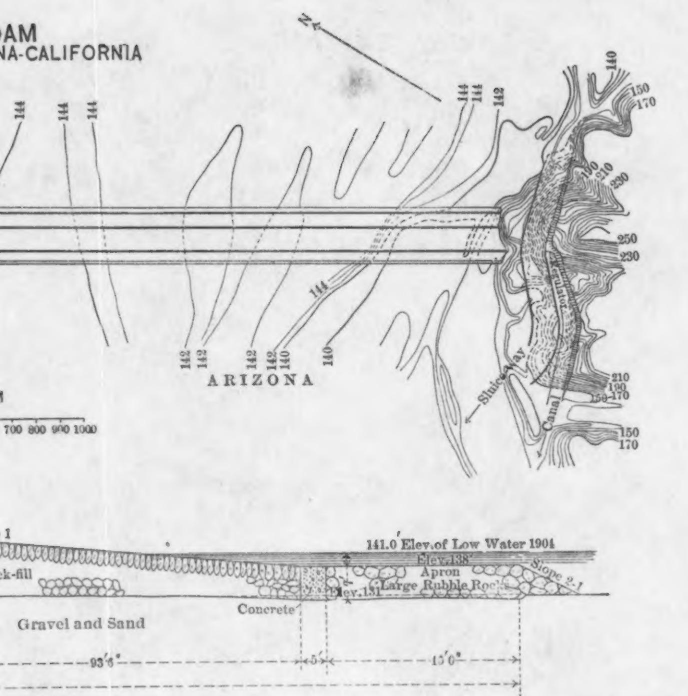
Laguna Weir.—The location and general design of this noted structure were determined by the character of the bluffs on each side of the last narrow point of the Colorado Valley, where they were almost a mile apart, and the fact that borings disclosed no bed-rock at reasonable depths in the river bed. Accordingly, it was decided to build a low, wide diversion weir of the so-called "Indian" type. The original design, as shown by Plate CIII, was constructed with practically only one modification, namely, the interchange of the principal diversion from the Arizona to the California side.

Purpose.—The purpose of this structure, primarily, was to provide for silting out the heavier particles carried by the river, during flood periods especially, where such deposits could be sluiced out from time to time and in such a way that river floods would certainly carry them down stream.

Consideration will be given later to the silt problem, but it may be said that the only way of keeping the large, heavy, valueless particles of silt from getting into the distribution system, and clogging it, is to provide a settling basin where the water for a short time will either be practically still or the velocity reduced to not more than 0.5 ft. per sec., with freedom from eddy currents. Such deposits may be removed either by sluicing out with large volumes of water at a high velocity, or by using pumps, dredges, or some other kind of machinery. It was estimated that in the main canal, originally de-

* Ninth Annual Report, U. S. Reclamation Service, Washington, 1911, p. 81.


LAGUNA DAM
YUMA PROJECT, ARIZONA-CALIF
COLORADO RIVER
Elev. of Crest 151′
Elev. of Apron 138′
Canal
Sluice-way
CALIFORNIA
PLAN OF DAM
SCALE OF FEET
Crest Elev. 151′
145.2′ Elev. High Water 1903
141.0′ Elev. Low Water 1904
Slope 2-1
24″ to 36″ Paving
Elev. 146′
Slope 12 to 1
Up Stream
Large Rubble Rocks
Rock-fill
Concrete
Sheet Piling
Gravel a
MAXIMUM SECTION ON A-B
SCALE OF FEET
Elev. 138.5′
Sluice-way Slope 0.00053
Crest Elev. 151′
Dam
Elev. 158
Elev. 138′
Regulator
Canal
West Abutment
PLAN OF HEAD-WORKS
SCALE OF FEET


DAM
NA-CALIFORNIA
N
ARIZONA
Sluice-way
Canal
141.0′ Elev. of Low Water 1904
Elev.138′
Apron
Large Rubble Rock
Slope 2:1
Concrete
Gravel and Sand
ON A-B
Slope 0.0006
Canal → Elev.141′
Regulator
Sluice-way
Elev.138′
Dam
East Abutment
RKS

signed to carry 1 600 sec-ft., the volume of wet silt to be removed therefrom daily was approximately 17 000 cu. yd. The sluice-way method of doing this means a higher initial cost and certainty of success, as compared with the very much lower cost, greater maintenance and operation charges, and somewhat less certainty of operation, for removal by machinery. The Laguna Dam (or rather weir) consists essentially of sluice-ways at each end of the structure, with a barrier between to hold up the water and afford a head for sluicing.

Sluice-ways.—The sluice-ways are controlled at their lower ends by large, vertical, steel-plate gates which are raised and lowered by electric machinery. The method of operation is to close the gates and cause the water in the sluice-ways above them to become practically still. The water thus held back quickly drops its heavier silt, while the canals are supplied through flash-board regulator gates in the outer sides of the sluice-ways, these gates being so long that a thin stream of water running over the tops suffices, and no water from near the bottom, where the sediment is greatest, is ever taken. From time to time, as may be necessary—and this varies greatly, depending on the stage of the river and the quantity of heavier silt particles carried—the gates are rapidly raised, and, with a fall of about 10 ft., the water rushes through the sluice-ways carrying away the silt deposits and dropping them a short distance below. During the annual and other floods, these deposits are taken up by the river and carried down stream. These sluice-ways are built through rock, their floor elevations being 13 ft. below the crest of the weir. They are lined and paved with concrete, and constitute a very massive and beautiful piece of work.

The Weir.—Between these sluice-ways the weir is built, the slope of the face being very flat, only 1 to 12, and capped with a concrete pavement, 18 in. thick, except a small portion which is paved with rough stones from 2 to 3 ft. thick. The crest was 10 ft. above the low-water mark at the dam when the structure was started,* and the top of the down-stream wall is 3 ft. below; thus the total fall of the face is 13 ft.

The weir was constructed simultaneously from each end, and a gap of 800 ft. was left in the center of the river channel. The original

* See discussion of variation in elevation of river bed at different times and different seasons, and the consequent low-water mark.

plan for completing this gap was by building upper and lower coffer-dams with piling, brush, and sand bags, but this was changed and finally the barrier rock fill dam, developed by the operations of closing the first and second breaks described later, was utilized. Before the central section was filled in, the sluice-way had been excavated and completed, the total capacity being more than enough to carry the low-water flow of the river. Rock was obtained in part from the excavation of the sluice-ways, and in part from the hills on each side; it was loaded on cars with derricks and steam shovels, and hauled by dinky locomotives to the various portions of the work. Coffer-dams made of quarry spoil were extended out into the stream, and inside these large pumps were used to clear of water. As much excavation as possible was done with teams and scrapers, and the remainder was taken out by suction dredges and pumps. Sheet-piling and the parallel concrete walls were then built, and the rock filling between was put in, followed up by the concrete surfacing. The actual quantities used exceeded the original estimates considerably; they are given in Table 6.

TABLE 6.

Rock excavation	444 640 cu. yd., or about	146% of estimates
Earth excavation	346 930 " " " "	123% " "
Rock in dam	375 018 " " " "	123% " "
Concrete*	76 066 " " " "	280% " "
Sheet-piling	82 779 lin. ft. " "	156% " "
Rock pavement	Insignificant—decrease.	100% " "

* In place of rock paving, the concrete surface was substituted.

On March 15th, 1905, bids for the construction of the Laguna Weir were opened, but those submitted were rejected and the work was re-advertised. Proposals were again opened on June 5th, and on July 6th, 1905, the contract was awarded at the following prices:

Rock excavation	$1.30	per cu. yd.
Earth excavation	0.30	" " "
Rock in dam	0.35	" " "
Concrete	4.00	" " "
Sheet-piling	0.40	" lin. ft.
Laying pavement	1.00	" sq. yd.

The contract required the work to be finished within 2 years, which would mean just at the time of the peak of the summer flood of 1907. These prices, on the estimated quantities, made the bid amount to

$797 650. There were seven other bidders, whose figures ranged up to $1 030 117.50. The Reclamation Service, under the specifications, supplied the cement to be used. On February 28th, 1906, the same firm was awarded the contract for furnishing and erecting the sluice-gates, regulator-gates, and operating machinery for the main sluice-ways and head-gates, the bid being $65 900. The contractors began work on July 19th, 1905, and a year later had completed 26.4% of the work.

As the quality of the rock obtained was much poorer than had been anticipated, the Board of Engineers of the Reclamation Service modified some requirements in the specifications and contract which resulted in increasing the contract price by $331 486, or to $1 129 136, and extended the specified time for completing the structure from July 19th, 1907, to January 19th, 1908. On January 23d, 1907, when about 34% had been completed, the work was taken over by the Reclamation Service direct. On July 1st, 1907, 52% of the work had been finished, a year later 77% was done; and it was practically completed in March, 1909, just before the summer flood of that year began.

The Reclamation Service gives the following costs* of the Laguna Dam and the sluice and regulator works:

Laguna Dam	$1 672 168.20
Sluice and regulator works	345 295.92

Other recent operations along the river have shown that a structure serving every purpose of the Laguna Weir could have been built by methods now well known at far less cost. The building of rock fill dams in the bed of such a stream as the Lower Colorado was considered impracticable until the work of re-diverting the river developed such method. However, it is now evident that it would have been far simpler, quicker, and cheaper to have developed rock quarries, thrown trestles across the bed of the stream, and dumped rock therefrom to form a wide rock fill dam, without any concrete walls whatever, and covered the top with concrete. There would be no difficulty in beginning the construction of such a dam in the center of the stream and causing the river itself to excavate its bed opposite the rock fill as the latter should be built forward. In this way the excavation would have been made to a little greater depth than the bottom of the concrete walls actually put in. The rock for such a purpose would by

* Ninth Annual Report, United States Reclamation Service, Washington, 1911, p. 82.

preference be graded, so that quarry spoil would in no way be objectionable, and rock material obtainable in the adjoining hills could be blasted out in large quantities, loaded with steam shovels, and consequently obtained and handled very cheaply. A structure having essentially the same top dimensions and surface covering, and extending deeper into the river bed than the existing one, would in this way have cost far less and be even more secure from failure. There would be practically no seepage through or under such a dam or weir, as the similar constructions, very much thinner and sustaining much greater heads, which closed the first and second breaks, seem to be absolutely water-tight.

To the cost figures should be added the proportional share of the total administrative and general expenses. Such administration figures are given as \$179 021.43, to which should properly be added at least \$75 000 of the item: "Preliminary surveys previous to selection of project—\$174 735.83," or a total of \$254 021.43. These are probably the approximate general expenses to be distributed over expenditures totaling \$3 717 472.71, or 6.89 per cent. On this basis, there should be added to the cost, for general expenses, \$114 243.24, or a total for the Laguna Dam proper of \$1 786 411.44, and to the sluice and regulator works \$23 583.71, making their total \$368 879.63, or a total of \$2 155 291.07, not including the loss of \$400 000 said to have been sustained by the contractor, which would raise the total to \$2 555 291.07.

The result is a magnificent and permanent head-works for taking water from the Colorado to irrigate by gravity about 75 000 acres of land in the Yuma Valley; and, at some future time, this structure may serve as well for diverting water to irrigate the entire Colorado Delta. Its very great cost, however, raises the question as to whether the silt problem could not have been solved in a more economical and equally satisfactory manner by pumping depositions thereof, in an enlarged section of the canal, back into the river, with suction dredges. This question cannot be determined until the maintenance costs of the sluice-ways and diversion weir are shown by experience, and the total costs and results of handling the silt with dredges, as is now being done at the California Development Company's head-works, have been ascertained for a considerable period.

Levee System of the Yuma Project.—Practically all the valley lands in the Yuma Project are subject to overflow, so that a general

and comprehensive system of levee protection is necessary. Fig. 5 shows this system, practically all of which has been completed. In general, the designs were for dikes 4 000 ft. apart along the Colorado and 3 200 ft. apart along the Gila, with a height of from 4 to 5 ft. above the high-water marks; as constructed, however, there are long stretches along the Colorado where the levees are only from 1 600 to 1 800 ft. apart.

The first levee construction was in accordance with the usual Mississippi River practice. The ground was cleared, stumps and roots were grubbed out, the base was plowed, and the levee was built with earth taken from borrow-pits on the river side. These borrow-pits were about 400 ft. long in the direction of the levee, with cleared traverses between about 12 ft. wide; 40-ft. berms; allowable depths of pits, 2½ ft. at the side nearest the levee and 3½ ft. at the farther side; levee top width, 8 ft; side slopes, 3 to 1 on the river side and 2½ to 1 on the land side. No muck-ditching was done.

The first stretch of levee constructed was 10 miles long, extending south from Yuma along the eastern bank of the river. In this section the current along the levee face was generally as little as would be expected anywhere on the project. Nevertheless, experience soon showed the desirability of an elaborate system of brush abatis work, a sample of what was put in here being shown by Fig. 1, Plate CIV. At many points where any considerable quantity of water had come against the face of the levee the borrow-pits had cut together, the traverses having quickly been cut through and the breach widened more or less seriously. As it was expected that the river would fill up these borrow-pits with silt in the first few floods, such a result was disappointing.

It seems that no trouble was caused by the absence of muck-ditch protection under the levee. This must have been due to the fact that the ground where the levee was located was uniformly favorable. In the fall of 1906, however, the levee system of the project was extended some miles southward along the river, and the flood which occurred on December 7th, 1906—which got under the newly constructed dikes on the west side of the river in many places and resulted in the second break or crevasse to the west—caused several breaks in this new section, due to the lack of muck-ditches in unfavorable ground.

Experience with the levees, including the effect of this last-

mentioned flood on the levee system of the project and on the levee work done on the other side of the river, caused a fundamental change in the design. In January, 1907, a Consulting Board of Engineers from the U. S. Reclamation Service was appointed to consider the matter of levee construction being done with money advanced by the Harriman interests on the west side of the river, and its recommendations are given later. Up to that date, 21 miles of levees had been constructed on the Yuma Project, extending from Yuma southward 15 miles and eastward along the south bank of the Gila 6 miles. All construction thereafter has been in accordance with the recommendations of this Consulting Board for the levees of the west side of the river, the essential features of which are "interrupted or checker-board borrow-pits" on the water side of the dikes, muck-ditches wherever test-pits show the necessity, and a large quantity of brush abatis work.

In 1907, a railroad track was laid in large part on the levee, from the Laguna Weir to Yuma on the California side of the river, chiefly for the purpose of hauling materials and supplies to and from the Laguna Weir. The Southern Pacific Company owns and operates this track as a branch line, thus serving an area which will be under intensive cultivation very soon, and greatly facilitating levee maintenance and repairs. Over this track a very large quantity of quarry spoil was hauled from the Laguna Weir construction work and used to blanket the river side of this levee to a point below where the swiftest water along its face is to be expected. None of the other levees of the project has any blanketing or any track on top.

Canal System of the Yuma Project.—Fig. 5 shows the general layout of the canal system of the Yuma Project as it is planned at present and in considerable measure constructed. The principal main canal is on the California side, and has a capacity of 1 700 sec-ft. The main canal on the Arizona side will irrigate only the land north of the Gila River. Water for irrigating the land lying east and south of the Colorado and below the Gila is to be carried under the river at Yuma in an inverted siphon, 1 000 ft. long, 14 ft. in diameter, about 50 ft. below the bed of that stream, and having an estimated capacity of 1 400 sec-ft. This siphon is now under construction. The original plan was to serve this territory with water taken from the Arizona end of the dam and carried across the Gila River in four rein-

FIG. 1.—TYPICAL ABATIS WORK ON LEVEES OF YUMA PROJECT, U. S. RECLAMATION SERVICE.

FIG. 2.—ORIGINAL INTAKE (CHAFFEY) GATE, IMPERIAL CANAL, COMPLETED IN 1901.

forced concrete tubes with a combined capacity of 1 300 sec-ft. and laid 3 ft. below the river bed. This crossing was abandoned because the difficulty of holding the Gila River banks at the ends of the underground siphon was considered too great. There is practically no danger of this kind in crossing the Colorado with the siphon, because of the little eminences of quite hard material which control the location of the river at this point.

The design of this siphon, the investigations of the material in which it is located, the first methods of construction used, the difficulties encountered, the changes in plans and methods, with the reasons therefor, the methods of doing the work finally adopted, and the time and cost figures, are all interesting in the extreme, but will not be given here for several reasons, chief of which is that, on the completion of the Yuma Project, it is hoped the work will be described at length in a paper by the Project Engineer, F. L. Sellew, M. Am. Soc. C. E., or some other engineer of the Reclamation Service. Only such general description is here given as seems desirable to make quite clear the effect of the project itself directly and indirectly on the irrigation of the delta.

The total acreage which will ultimately be irrigated by the Yuma Project is given in the reports of the Reclamation Service as 90 160. This includes 17 000 acres of mesa lands which lie too high to be reached by gravity from the principal canal system. It is intended to develop 1 000 h.p. at the drop in the main canal, and with this to operate pumps to raise water for the higher distribution systems.

At present the main canal from the Laguna Weir on the California side down to the California shaft of the river crossing at Yuma is under construction, and quite a little main and some lateral canals lying between these points and behind the line of the levees on the west side of the river have been completed and are in use. Such canal construction, and particularly the checks, head-gates, etc., are models of their kind, being of concrete and of the latest and most approved type. Up to the end of 1907, there were no canals on the California side of the river; indeed, practically all the area was contained in the Yuma Indian Reservation, which has since then in part been apportioned to individual Indians and in part, 6 500 acres, on March 1st, 1910, opened to entry and quickly taken up by white settlers. Two weeks later water was turned into the reserva-

tion canals, and rapid progress is being made in developing the region. On the east side of the river below Yuma about 8 000 acres are being irrigated through small canal systems which have been in operation for a long time and were taken over by the Reclamation Service since the creation of the project. The total acreage of the project to which water could have been supplied was about 16 000 acres, while about 10 000 acres were actually irrigated during the season of 1911.

Drainage System of the Yuma Project.—As has been said, a large acreage of the Yuma Project is subject to annual overflow, and lies behind the levees. The water-table throughout practically the entire region rises and falls with surprising rapidity during all floods which are long in passing. Thus it is that during May, June, July, and August particularly, the water-table rises so near the surface as to result in rather high alkalinity in the soil. The river water which will be applied for irrigation also carries a considerable, though not serious, quantity of soluble salts. Evaporation takes place from land surfaces very rapidly in such a hot country, and when water is on the surface, or approaches so near it that capillarity makes connection between the water-table and the surface of the ground, the quantity evaporated is excessive, and the salts contained are left behind, largely in the top layers. Therefore, efficient drainage is very important. It is made even more necessary because the rainfall is really inappreciable, having been less than 3 in. per annum for the past 15 years, and causes very little leaching and washing away of alkaline depositions. In passing, it is important to say that, very fortunately, the alkali of the valley lands is peculiarly a surface accumulation, often being confined to the very upper layers, usually to the first 2 ft. in depth, and seldom being found at depths exceeding 6 ft.

The Yuma Project, therefore, includes plans for an elaborate and efficient system of drainage canals which will be doing the maximum amount of work during the annual summer floods of the Colorado. It is planned, where necessary, to pump such drainage water over the levees and back into the river. This drainage system has not been constructed, and indeed the detailed plans may not yet have been worked out, but it is desired to state here that arrangements have been made for drainage in the Yuma Project, and results along that line must be obtained in the Imperial Valley sooner or later.

Imperial Valley Irrigation Project.

In 1893, Mr. Rockwood found himself in possession of much engineering and other information regarding the irrigation of the Colorado Desert with the water of the Colorado River, in lieu of salary for a considerable time as Chief Engineer of the Colorado River Irrigation Company, and had a firm conviction of the project's possibilities. For more than 7 years he endeavored to finance the work, both in the United States and abroad. Many people have suggested the irrigation of the Colorado Desert, as already mentioned, but Mr. Rockwood and associates actually brought it about. The very interesting history of the enterprise,* unfortunately, is accessible to relatively few people. In spite of his later mistakes, Mr. Rockwood is certainly entitled to much credit and reward for his efforts, which, practically speaking, were finally crowned with complete success.

Engineering Features.—The engineering features of irrigating the Imperial Valley from the Colorado River can now be much better understood than was possible in 1900. The experience of 10 years is always of value, and was particularly so in this case. The fall of the ground was known, and to divert the water and conduct it to the broad, ideally lying tracts of land to the west of the sand hills was obviously practicable. There were, however, two especially serious problems: the danger of diverting water from a wide, erratic stream flowing through a shifting channel along the top of a ridge of loose alluvial silt; and the difficulty of keeping open canals which carried water so heavily charged with silt.

Diversion.—The impossibility of properly financing the enterprise absolutely forced the abandonment of the idea of diversion at The Potholes, with its opportunities for settling basins and sluice-ways to care for the silt *en route,* and made the diversion at the rocky point of Pilot Knob practically unavoidable. It was always the idea to have a head-gate founded on solid rock. At the last, it was found impossible to obtain the money, even for this construction, but the diversion point was located there, with the intention of utilizing this rocky point of Pilot Knob for head-works, in the very near future, when the financial status of the company might permit.

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* "Born of the Desert," by C. R. Rockwood, in the Second Annual Magazine Edition, *Calexico Chronicle*, Calexico, Cal., May, 1909.

Flood Protection.—It does not seem to have been realized, at the time, or indeed by any one until the diversion into the Salton Sea was actually an accomplished fact in 1905, that there was any really appreciable danger to the Imperial Valley by flood-waters from the Colorado. The writer hopes especially, that the discussion will bring out any contradiction of this statement which may be successfully maintained. Of course, it was known that large quantities of water had been carried through the New and Alamo Rivers into the Salton Sea in 1891, and also by the New River earlier, especially in 1862, but the channels had not eroded to any marked degree at the gathering ground along the Colorado River bank, but, on the contrary, had automatically closed. Instrumental data regarding that portion of the delta cone which is subject to overflow were entirely lacking, and indeed, little other reliable information about the region was available. It was planned to build levees along the river side of the canal with the material taken from the latter, but the purpose of these levees was to protect the canal itself from danger, and not to keep the flood-waters which might enter this waterway from enlarging it to a dangerous degree. Of course, any risk of the river being diverted into the Salton Sink, and soon inundating the entire Imperial Valley, involves the same risk to the irrigation project as such. Otherwise, such risk should obviously not affect an irrigation company in any way, unless its operations and constructions have an appreciable effect on such river diversion.

Silt.—With this means of diversion it was necessary to let the silt-laden river water enter the canals directly, and depend on keeping them open by dredging, erosion, etc. The chief difficulty obviously must occur in the first stretches of the waterway, due to the rapid deposition of the heavier or sandy particles of silt which the river water carries during flood stages of excessively high currents, and which drops down almost at once when the velocity decreases to, say, 3½ ft. per sec. After such clarification, it is possible to design and operate canals in the Colorado Delta, as well as in India and elsewhere, so as to insure the carriage of the remaining finer silt into the smaller laterals and to the irrigated land. The first mile of the canal, therefore, was designed with a large cross-section so as to secure the deposition of this heavier silt there, where it could be removed by dredges.

Alkaline Lands.—From a farming point of view, a difficulty which was not given very serious consideration was the relatively high alkalinity in the upper layers of the soil throughout practically all the Imperial Valley. Wherever water came in contact with the ground, it was observed that vegetation at once sprang up like magic, and it was assumed, from this and from the obvious methods of its occurrence, that the soil must be exceedingly fertile and admirably adapted for general agricultural purposes. In one sense, a very serious mistake in this way was not made, for agriculture of almost unparalleled success has been followed for the past 10 years, with only at rare intervals a very slight thinning of crops indicating the need for proper drainage and the reduction of the excess of alkalinity.

In 1893 the Director of the Agricultural Experiment Station at the University of California was asked to investigate the agricultural possibilities of the land in the Imperial Valley. At that time it was proposed to provide an expedition properly equipped in order that the Director, Professor E. W. Hilgard, the great American authority on soils, might explore the region personally. The financial difficulties of the Company prevented carrying out the plan at the time, but a few samples of water from the lakes and of soil taken superficially, proved that the latter were very similar to that of the immediate bottom of the Colorado River, which previous analyses had shown to be of extraordinary intrinsic fertility.* In 1896 and 1897, some additional samples of soil and water were sent for examination. These corroborated the previous conclusions, but showed that a considerable quantity of alkali salts was present in the soils as well as in the waters, and thus indicated the desirability of a thorough examination of the region, from the soil standpoint. The subsequent soil investigations in the Imperial Valley and their effect on the fortunes of the region will be considered later.

Drainage.—While the country as a whole lies ideally for irrigation and ordinary irrigation water drainage, the natural waterways are so far apart and so small and ill defined as to make the construction of an efficient, comprehensive drainage system almost as difficult and expensive as the irrigation canal. In the engineer's report to the Colorado River Irrigation Company, it was stated that the construction of a drainage system (while almost as expensive as the proposed

* Report, Agricultural Experiment Station, University of California, 1882.

irrigation system) was essential, but some years later, when the work to be done was trimmed to the lowest practical minimum, it was decided that a general drainage system was not immediately necessary and possibly might never be required. This latter opinion was not as radical as might at first be assumed, because, even to-day, there are probably not more than 5 miles of drainage ditches in the valley. It is being realized in a general way that at some time provision for drainage must be begun, and within the next few decades doubtless a fairly comprehensive plan will be developed. The diversion of the Colorado into the Salton Sea in 1905-06 resulted in eroding the beds of the Alamo and New Rivers into deep wide channels which will be the controlling features in the design of the ultimate drainage system for the valley, and thus produce a benefit which in the end must certainly exceed the total damages resulting from such diversion.

Climate.—The climate of the region, with its long, hot, dry summers, is peculiarly favorable to agricultural luxuriance. Thus it is that here the very earliest grapes, fruits, and vegetables are produced for the United States market, with the consequent advantage of commanding the highest prices. This is notably true of the Imperial Valley cantaloupe, now famous all over this country, and of the early grapes, asparagus, etc. On account of the very low humidity and gentle winds which blow much of the time in hot weather, the sensible temperature—which is indicated by the wet-bulb readings and gives the measure of heat felt by the human body—is much less than the actual temperature as measured by the dry bulb. It is conservative to say that a temperature of 110° in Imperial Valley is not more uncomfortable than 95° in Los Angeles or 85° in the more humid sections of the Eastern States. Furthermore, the nights are always cool, the low humidity resulting in rapid and large daily temperature variations.

At the same time, the heat in the Colorado Desert and at Yuma was proverbial, and one of the difficulties which the project had to encounter was the supposedly frightfully hot summers; indeed, the project would otherwise have been financed very much earlier. Since the control of the diversion canal was lost in 1905, the impression has become general that the project of irrigating this region was rejected by capitalists as involving too great engineering risks. As a matter of fact, the chief difficulty was the fear that the torrid climate would render colonization very difficult.

The International Boundary Line at the Sand Hills.—Perhaps, everything considered, the location of the International Boundary Line and the Sand Hills which lie to the west of Pilot Knob and overlap into Mexico for several miles, constitute the most important features of the irrigation, and protection from inundation, of the Imperial Valley. It is this which makes it impossible for the people of American Imperial Valley to organize to protect themselves under the laws of California. The menace is entirely on Mexican territory, and, apart from the difficulty of dealing with the problem as one of engineering and statecraft, is the worst feature of all, namely, the seeming injustice of compelling American citizens to protect their homes against a menace originating entirely on foreign soil.

Aside from the danger of the diversion of the Colorado to the west and into the Salton Basin, the important result of the present location of the International Boundary Line is that, practically speaking, water cannot be taken from the Colorado River and carried in canals lying wholly on American soil to the areas in American Imperial Valley susceptible of irrigation by gravity. It could be done, but it would require approximately 12 miles of a closed conduit running under the sand hills and costing at least $10 000 000, a sum practically prohibitive.

Water Rights.—Due to the divided authority of the National and State Governments with respect to permission for taking water from the Colorado River as a navigable stream, water appropriation notices then, as now, had to be posted and filed, under the laws of the State of California, and arrangements had to be made with the United States War Department as well, if such diversion interfered with navigation. It appears that no attempt was made to obtain permission from the War Department for taking water from the river, because it was almost impossible to cause any "interference with navigation." This failure to secure permission from the War Department, however, had a very serious result later.

Ideal Plans.—The ideal way to carry such a project through is now quite obvious. All the engineering features should have been carefully worked out, elaborate soil surveys should have been made by well-recognized authorities, and experimental farms should have been established. The irrigation system should have been built in sections and colonized before additional areas were covered by canals. Water

rights, entirely free from any possibility of attack, should have been obtained. In the light of experience, the writer believes that, by all means, these should have been obtained from the Mexican Government, and the diversion should have been made on Mexican soil, or the development should have been made under the Carey Act.

Dealings with Mexico would have meant the abandonment of the idea of diversion works founded on solid rock, but a structure with a wooden caisson foundation extending under the gates proper and the wing-walls would have been just as safe as the concrete head-gate actually put in later, and would have cost little more money, if indeed as much.

The ownership of all private interests in the Salton Sink ought to have been acquired, and such permission obtained from proper Government authorities that this naturally depressed basin would ever be available without question as a receptacle for the seepage, drainage, and waste water from the irrigated lands and canals. Data as to silt deposition and the cost of removing it from canals and intakes should have been obtained from experiments carried out on a commercial scale. Various details of the project, in short, should have been worked out very carefully and adhered to.

However, as in many irrigation and other projects in the West, the garment had to be cut according to the cloth. The sum total of events resulted in carrying out the project along lines which were far from ideal, but which later proved to be possible of execution with a remarkably small amount of money, everything considered.

The California Development Company.

The first practical step toward the actual irrigation of Imperial Valley was the incorporation of the California Development Company, under the laws of New Jersey, on April 26th, 1896. After two years of vain endeavor to obtain permission from the Mexican Government for the American corporation to hold land and acquire rights of way for the main canals into American Imperial Valley, it was found necessary to form also a Mexican corporation. The California Development Company has a capital stock of $1 250 000, divided into 12 500 shares of $100 each; the Mexican Company—*La Sociedad de Riego y Terrenos de la Baja California, Sociedad Anonima*—has a capital stock of $62 500, all of which is owned by the California De-

velopment Company. Hereafter in this paper the California Development Company will be referred to as the C. D. Co., and the subsidiary Mexican corporation as the Mexican Co.

The general practice throughout the West was, and still is, the sale of the "water right" to settlers at a definite price per acre—usually the right to buy water thereafter at specified prices. The arrangement adopted in this case was the formation of mutual water companies which would receive water wholesale and distribute it to their stockholders, the capital stock of such mutual companies constituting the water right.

Organization Under the Carey Act.—It would undoubtedly have been much better if the desert land in the United States had been segregated, and if the project, as far as American territory was concerned, had been carried out under the Carey Act. This Act, however, had not been passed when the original investigations were made, and, when financial arrangements were concluded, the California Legislature had adjourned and would not meet for nearly two years. Such delay was deemed too great.

Water Appropriations.—Water filings were made on April 25th, 1899, on the right bank of the Colorado River about 3 000 ft. above the International Boundary Line, by Mr. C. N. Perry, on behalf of the C. D. Co., appropriating 10 000 sec-ft., of the flow of the Colorado River to be used for the irrigation of American lands in the Imperial Valley. No serious attempt was made to obtain water rights in Mexico—in Mexican territory there was no chance to found diversion works on rock, and money for the first work of promotion would have been difficult to obtain with a projected intake in that country.

Rights of Way.—The C. D. Co. purchased 316 acres of patented land along the river just north of the International Boundary Line, and these included the rocky point of Pilot Knob; and the Mexican Co. acquired 10 000 acres in Mexico, belonging to Gen. Guillermo Andrade, and lying generally south of the Boundary Line, as shown in Fig. 7, together with the bed of the Alamo River, which extended beyond the boundaries of this tract. In the American Imperial Valley (all the land belonging to the Government) rights of way could not be purchased outright, but easements therefor were easily obtained as at present by application to the Secretary of the Interior, accompanied by maps and descriptions of the proposed constructions. All rights

of way and property required for the construction of the project were thus arranged.

Contractual Relation of the C. D. Co. and the Mexican Co.—The two companies entered into a contract by the terms of which the C. D. Co. turned over to the Mexican Co. all the water to be diverted from the Colorado River by the former where the canal crosses the International Boundary Line at Algodones; the Mexican Co. agreed to deliver water to water users in Mexican territory as required and the remainder of the supply—the larger part by far—to the American water users at points on the International Boundary Line from 40 to 50 miles west of the river, and, from the water users of both countries, to collect for the water furnished, on a quantity basis; the C. D. Co. agreed to build, maintain, and operate all the Mexican Co.'s irrigation construction in Mexico; the Mexican Co., in consideration thereof, agreed to pay the C. D. Co. all sums received by the former for water rights, water stock, and water rentals from water users in the United States. These agreements were limited to water for lands which were irrigable by gravity from the system of canals beginning at the head-works constructed. It was stipulated, further, that no contract should be entered into with the Mexican Co. giving any person or corporation superior right over any other water user by reason of priority in date of contract or otherwise, and that the C. D. Co. should not be responsible for failure to deliver water to the Mexican Co. from any cause beyond its control, although admitting obligation to use due diligence in protecting canals and maintaining the flow of water therein.

By this arrangement, the Mexican Co. retains the money received from the water delivered to Mexican water users, and is put to no construction, maintenance, or operation expense whatever. This arrangement, however, is not as advantageous as at first appears, because the gross annual water rentals from Mexican water users did not amount to $10 000 gold per annum until the beginning of the ninth year, while the right of way contains at least 2 500 acres of land and includes 50 miles of the Alamo River channel, which is utilized as a main canal. It will be a number of years yet before the receipts of the Mexican Co. will be sufficiently large to make the contract an unusually profitable one.

Mutual Water Companies.—Next to the general plan of arranging

to require the purchase by settlers of the water right usual in such cases, the fundamental idea was delivery of water to mutual water companies instead of individuals, the mutual companies to be operated by the holders of stock, namely, the farmers in their respective districts. The various mutual companies thus run their own local affairs and join together, through the C. D. Co. and the Mexican Co., in a community main canal leading from the river to the settlement west of the Sand Hills.

Triparty Contracts.—The mutual water companies required the construction of a distribution system, and ought or ought not to have paid a bonus for the contract to receive water at the International Boundary Line, depending entirely on the conditions under which the water should be delivered and the price to be paid for it. A triparty contract was entered into between the Mexican Co., the C. D. Co., and each of the mutual water companies, under the terms of which the Mexican Co. agreed to supply water to the mutual water companies "on demand" and at definite points on the International Boundary Line in the Imperial Valley for 50 cents per acre-ft., to be used only on lands within the respective districts; provided, however, that the aggregate quantity of water necessary to deliver under the contract should not exceed four times the number of acre-feet per annum that there were shares of capital stock in the mutual company; the mutual company agreed to order and pay for at least 1 acre-ft. of water each year for each share of its stock sold and located, regardless of its use by the mutual company or by its stockholders; the C. D. Co. agreed to build the distribution system of the mutual company and to maintain certain definite portions of the canal thereof perpetually, reserving the right to develop and use the water-power that might be obtained from the waters running through any of the canals, including those of the mutual company; a provision was made that at the end of 3 years the loss of water to the C. D. Co. in evaporation from the canals of the mutual company should be determined, and such an extra allowance of water be supplied, as so determined, to the end that only the net quantity reaching each half section of land should be paid for; and the mutual water company turned over all its capital stock to the C. D. Co. and agreed to locate such stock on any lands within the exterior boundary lines of its district on order of the C. D. Co. The C. D. Co. sold the capital stock of these vari-

ous mutual companies to the settlers, and with the proceeds built the main canals in the United States, the canal system in Mexico which belongs to the Mexican Co., and the distribution systems which became the properties of the various mutual water companies.

There were eight of these triparty contracts; they were essentially similar, though no two were exactly alike in every detail. The contract between the Mexican Co. and the C. D. Co., and the triparty contract as just outlined, together with the by-laws of the mutual companies, show the contractual relation of the water user to the organizations on which he depends for water. These by-laws, in general, provide that each share of stock shall represent the right to purchase water for the irrigation of 1 acre of land; that stock issued shall have written on its face a description of the land on which it is located; that no stock shall be located on any lands outside those described in the articles of incorporation; that one share and no more shall be located on each acre of land which can be served by the ditches of the company; that owners of stock issued but not located shall not be entitled to receive any water represented thereby, but shall, nevertheless, be liable for all assessments, the same as other outstanding stock of the company; that the shares may be transferred; and that acceptance by any stockholder of a certificate of stock shall be considered as a ratification by him of any and all contracts between the mutual company in question and the C. D. Co.

The inter-relations of the water users and the various corporations have been given in detail because of a general impression that the plan was devised for the purpose of taking advantage of the settlers. In its operations it has resulted in no unfairness of any importance to any of the parties concerned. Considering all the circumstances, the prices charged for water rights were very low—$8.75 at the beginning, up to $20 at present, and averaging $12 per acre as the total cost to the settler, on easy terms—and the total annual water rental from the water users in the valley will not suffice to pay maintenance, operation, and general expenses, properly figured, until such time as about 700 000 acre-ft. of water are sold annually. At the end of 9 years the sales have not yet reached that figure. Fig. 4 shows the boundaries of the lands of the various mutual water companies in the valley and under whose distribution systems lie all the lands which are as yet irrigated.

TABLE 7.—COMPARATIVE STATEMENT OF EARNINGS AND EXPENSES OF THE CALIFORNIA DEVELOPMENT COMPANY, FOR NOVEMBER, 1909.
(Property on a seriously deteriorating basis.)

	November, 1909.	5 months, ending November 30th, 1909.
EARNINGS:		
Water sales	$13 906.20	$93 236.75
Water-power royalties	418.90	1 772.80
Rent, buildings and other property	82.05	488.51
Miscellaneous earnings	66.45	420.68
Gross earnings from operation	$14 473.60	$95 918.74
OPERATING EXPENSES:		
Maintenance, canals and structures:		
Superintendence	$163.83	$641.63
Maintenance and cleaning canals	10 588.76	47 130.74
Bridges	223.04	408.88
Canal structures	901.70	2 444.57
Buildings, fixtures and grounds	824.81	3 192.44
Total	$12 702.14	$53 818.26
Maintenance of levees:		
Superintendence	....	$210.00
Patroling	$102.55	377.87
Roadway and track	126.12	1 034.01
Telephone and telegraph lines	84.87	217.04
Buildings, fixtures and grounds	....	44.37
Total	$313.54	$1 883.29
Maintenance of equipment:		
Vehicles	$85.23	$576.47
Grading implements	....	14.23
Corrals	29.01	525.56
Machinery	....	533.45
Shops	34.71	155.25
Automobile	135.36	1 115.04
Motor cars	83.34	154.46
Dredges	281.42	938.18
Total	$649.07	$4 012.64
Distribution of water:		
Superintendence	$163.70	$997.22
Zanjeros	760.00	3 858.63
Calibration and water measurement	21.52	365.00
Telephone and telegraph lines	163.40	815.53
Damages	....	50.00
Total	$1 108.62	$6 086.38
General expense:		
Salaries and expenses, general offices	$2 981.66	$15 444.09
Office expenses	332.37	1 515.04
Law expenses	618.28	3 899.92
Stationery and printing	132.36	684.86
Other expenses	107.40	459.67
Total	$4 172.07	$22 003.58
Total operating expenses	$18 945.44	$87 804.15
Net earnings	*$4 471.84*	$8 114.59
Taxes	$197.76	$1 134.88

Operation of Triparty Contract.—For 3½ years the writer was General Manager for both the C. D. Co. and the Mexican Co., and handled all matters between these companies and the various mutual water companies. During the latter portion of that time, the protection of the Imperial Valley from inundation by the Colorado had become quite as important as its irrigation, and, for this protection, of course, no provision was contemplated in these contracts. Except for that, the arrangement proved to be very satisfactory, and produced a smooth and comfortable relationship unusual in irrigation enterprises. As a result of litigation, however, the Supreme Court of California has just declared the whole scheme practically illegal, the text of the decision not yet being available. The Imperial Irrigation District was created several months ago, and the directors thereof have decided to take over only the functions which the C. D. Co. and the Mexican Co. now perform, and will not interfere in any way with the mutual water company plan of organization, or the water companies themselves.

TABLE 8.—STATEMENT OF EARNINGS AND EXPENSES OF LA SOCIEDAD DE IRRIGACION Y TERRENOS DE LA BAJA CALIFORNIA.

	November 5th, 1909.	5 months, ending November 30th, 1909.
GROSS EARNINGS	$641.20	$5 127.39
OPERATING EXPENSES:		
Distribution of water	0.00	0.00
General expense:		
Salaries and expenses, general officers and clerks	421.54	2 073.85
Office expenses	64.07	341.32
Law expenses	223.99	696.64
Stationery and printing	39.55	101.21
Inspection fund (Mexican Government)	150.00	750.00
Other expenses	231.25	663.47
Total	$1 130.40	$4 626.49
Total operating expenses	$1 130.40	$4 626.49
Net earnings	*$489.20*	$500.90

Imperial Land Company.—The parties who were induced to back the enterprise financially were afraid of the colonization end, and would have nothing whatever to do with it. Accordingly, it was neces-

sary to form a colonization company—the Imperial Land Company—which was incorporated under the laws of California in March, 1900, and consisted in part of some of the promoters of the C. D. Co. and in part of other people. This corporation contracted to do all advertising and colonizing and sell all water stock in consideration of having the exclusive privilege of town sites and a commission of 25% on water stock sales. By using Government land scrip, this company obtained immediate ownership in fee simple of tracts of land in various parts of the valley and subdivided them into town sites. These town sites were covered with water stock in order to obtain water for domestic and municipal use through the assistance of the mutual companies, because no wells, except some very deep and unsatisfactory ones quite recently sunk on the east side of Imperial Valley, have ever been possible for domestic supply. The Imperial Land Company thus established the town sites of Mexicali, in Mexico, and Calexico, Heber, Imperial, and Brawley, in California. The other town sites—El Centro, the county seat, Holtville, Seeley, Dixieland, and several smaller places were platted and put on the market by other parties.

TABLE 9.—OPERATING EXPENSES OF CALIFORNIA DEVELOPMENT CO., JANUARY 1ST, 1908, TO MARCH 31ST, 1909.

	1908, 12 months.	1909, January, February, and March.
Maintenance, canals and structures	$71 419.94	$18 177.32
Maintenance, levee	10 260.35	647.24
Maintenance, equipment	18 528.69	4 182.21
Distribution of water	15 613.42	4 559.10
General expense*	75 162.82	12 277.76
Construction of canals	73 765.12	27 359.47
Construction of levees	32 303.09	32 297.84
Totals	$297 053.43	$99 500.94

* Of this sum, $30 665.28 was litigation expenses and costs.

During 1911 the total net deliveries of water to the mutual water companies in the United States were 597 178 acre-ft., or $298 490.98.

This colonization company in general was successful, but not to the extent which would be expected, considering the unprecedentedly rapid settlement of the region, and the contract was certainly a fair one to the C. D. Co., up to the time of its abrogation in 1906. Water stock was sold to the settlers for small cash payments and notes payable in

five yearly settlements at 6% interest, such notes being secured by a mortgage on the water stock purchased. Many of the settlers had scant means and only a filing right on the land, so that the water stock was not made appurtenant to the land, but left as personal property. The initial payment went to the Imperial Land Company, and was by it used for advertising and other essential purposes, the collateral notes and mortgages secured by the water stock being taken by the C. D. Co.

TABLE 10.—AVERAGE DIVERSION AND DELIVERIES OF WATER BY THE CANAL SYSTEMS OF THE C. D. CO. AND THE MEXICAN CO. FOR THE WEEK ENDING JANUARY 19TH, 1912.

Gauge at Yuma		15.3 ft.
Gauge opposite intake		105.9 "
Elevation of bottom of diversion gate		98.0 "
Average flow of Colorado River at Yuma		4 000 sec-ft.
Diversion from Colorado River at Andrade		1 559 " "
Used in Mexico	37 sec-ft.	
Used in United States	894.6 " "	
*Wasted at Rositas waste-gate	321.3 " "	
Total		1 252.9 sec-ft.
Total loss, Andrade to Sharps		306.1 sec-ft.

* 171.1 sec-ft. of this passed through the plant of the Holton Power Company *en route* to this waste-gate for developing electrical energy.

This loss equals 19.6% in about 46 miles of main canal, chiefly the bed of the old Alamo River, or 0.43% per mile on the average—an extremely low figure.

Management of the C. D. Co.—Delta Investment Company.—Until the water rentals became of importance, these collateral notes and mortgages constituted the only receipts of the C. D. Co., and these assets were looked on with considerable suspicion by the financial institutions of Los Angeles. Nevertheless, they were taken as collateral at about 25 cents on the dollar until the merit of the entire enterprise was rendered questionable in various ways, as explained later. When this occurred the Delta Investment Company was formed—in the fall of 1901—with assets consisting solely of C. D. Co. and Imperial Land Company stock contributed by the wealthier people of the enterprise, whose confidence was waning. This company was given a contract to take over all the C. D. Co.'s bonds at 50 cents on the dollar; and all its collateral notes and mortgages at the same discount. By this arrangement, the Delta Investment Company faction absolutely controlled the C. D. Co., although the amount of the C. D. Co. stock held by it was much less than a majority.

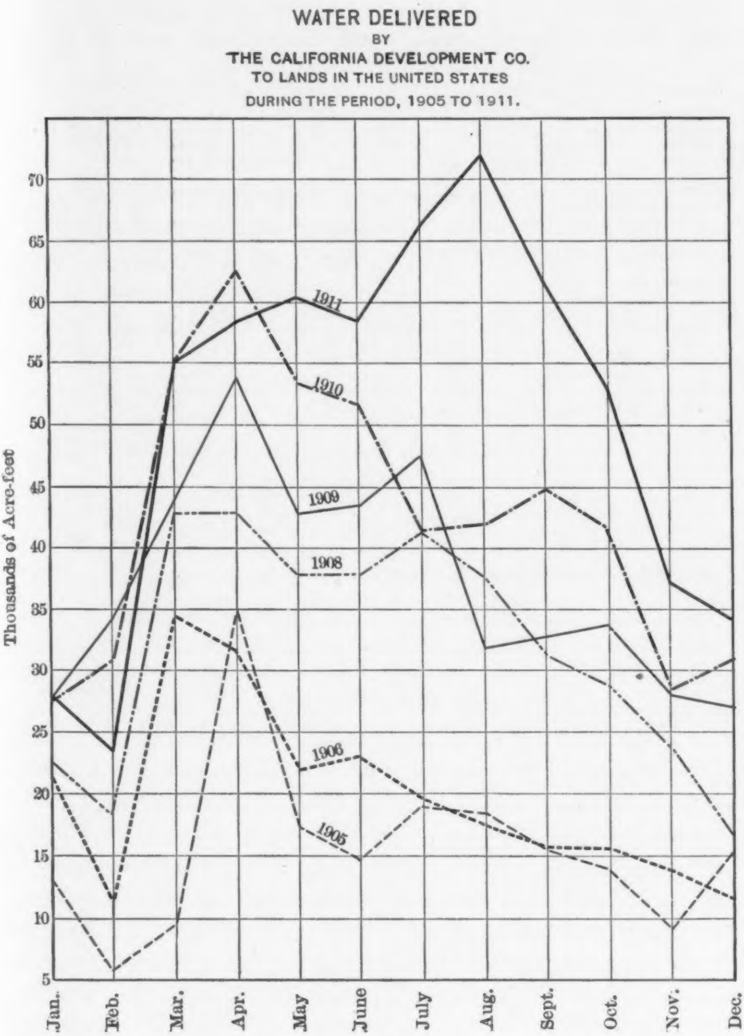

WATER DELIVERED
BY
THE CALIFORNIA DEVELOPMENT CO.
TO LANDS IN THE UNITED STATES
DURING THE PERIOD, 1905 TO 1911.
Thousands of Acre-feet
70
65
60
55
50
45
40
35
30
25
20
15
10
5
1911
1910
1909
1908
1906
1905
Jan.
Feb.
Mar.
Apr.
May
June
July
Aug.
Sept.
Oct.
Nov.
Dec.

FIG. 8.

TABLE 11.—ANNUAL EXPENDITURES OF IMPERIAL WATER COMPANY No. 1 FOR 1911.

Capital stock = 100 000 shares, all of which have been sold, and are located on 100 000 acres of land. Total length of canals = 373.25 miles.

Maintenance:		
Superintendence	$7 000.00	
Engineering	1 600.00	
Corral	3 805.11	
Automobile	500.00	
Shops	2 463.13	
Materials and supplies	22 046.11	
Labor, men and teams	75 887.41	
Damages	645.91	
Muskrats, bounty at $1 each	492.00	
		$114 439.67
Operation:		
Superintendence	$3 815.74	
Engineering	168.81	
Zanjeros	22 609.02	
Corral	3 192.36	
Automobile	476.39	
Materials and supplies	1 892.98	
Telephone	260.16	
Water meters	260.47	
		32 675.93
General Expense:		
Salaries	$6 410.02	
General expenses	2 971.13	
Printing and stationery	439.94	
Taxes and insurance	956.59	
Furniture and fixtures	520.45	
Legal expenses	10 889.38	
		22 187.51
Imperial Water Company No. 1, expense		$169 303.11
Water Bought (from the C. D. Co.) 305 183 acre-ft., less 10% allowance for seepage and evaporation, at 50 cents per acre-ft., on net amounts		137 332.50
*Total expenditures**		$306 635.61

* The expenses of the company were almost exactly $1.70 per acre, and the water rentals paid to the C. D. Co. $1.37 per acre. The total cost to the farmers, therefore, averaged $3.05 divided by 2.747, or $1.11 per acre-ft.—a very low figure for water in California, where the "water right" averages $12 per acre, or indeed much more.

It must be admitted that the Delta Investment Company took over such securities at a larger price than could have been obtained from any other source. Nevertheless, the securities were really good, everything considered, and quite a few large and apparently strange and dishonest transactions were made between the two corporations resulting to the great benefit of one faction of the C. D. Co. at the expense of the other. Money was forthcoming for construction purposes, but was costing the C. D. Co. $2 for every $1 obtained. The result was that in a couple of months serious dissensions arose, and in February, 1902, an adjustment was made cancelling the contract with the Delta Investment Company and eliminating the original financial backers from further connection with the enterprise. March 1st, 1902, there-

fore, found the C. D. Co. with all its bonds gone, its collateral notes and mortgages largely depleted, no money in the treasury, and deeply in debt. Shortly afterward actual results from farming under the project were so reassuring that the company was able to borrow $25 000 from the First National Bank of Los Angeles and begin afresh.

The contract with the Delta Investment Company was a serious thing for the C. D. Co., but, to be perfectly fair in presentation, it must be borne in mind that the financial interests backing the enterprise had their confidence in the project so violently shaken by advance rumors of an adverse Government soil report (to be discussed later) that they felt justified in trying to get back all that might be possible from the wreckage.

With the exception of the arrangement with the Delta Investment Company, no proper criticism can be made of the handling of the finances of the whole irrigation project, as far as any of the promoters of the irrigation company are concerned. The writer has had opportunity and occasion to investigate thoroughly the relationship of all the corporations, and in common fairness must state that, while the deals back and forth were many and diverse, they were otherwise with very few exceptions reasonable and fair, when the circumstances and reasons which produced them are given the proper weight. Furthermore, the general aims and plans which the company practically succeeded in carrying out do not merit any more criticism than those of the average Western irrigation project, if indeed as much. Had the break in the Colorado River never been allowed to get beyond control—and it never would have happened, in spite of all obstacles, had the loan of the Southern Pacific Company (referred to later) been arranged 6 months earlier than it was—the C. D. Co. would undoubtedly have proved to be one of the most successful private irrigation enterprises throughout the entire land.

Colorado River Land Company.—It is well at this time to mention the Colorado River Land Company and the New Liverpool Salt Company. The former is a corporation consisting principally of Southern California stockholders, incorporated under the laws of Mexico, and owning about 1 000 000 acres south of the International Boundary Line and west of the Colorado River. It owns all the Colorado River Delta west of the river in Mexico except 162 000 acres, the location of these holdings and those of other important Mexican land owners

being shown on Fig. 4. The existence and operation of this corporation have lately become important as being the agency through which the United States Government has handled the river control work recently done by it. The company will hereafter be referred to as the C. M. Co., as it is locally called.

The New Liverpool Salt Company.—This corporation was organized many years ago for the purpose of obtaining salt from the deposits in the bottom of the Salton Sink, and began operations in 1884. In 1904 its plant was reasonably satisfactory in its details and had a capacity of 1 200 tons of salt per month. The actual value of the plant and the salt beds, taking into consideration the excellent quality of the salt,* the conditions under which the Company operated, and the competition it had to meet, is of course impossible to determine without access to the company's records. It appears, however, that negotiations at that time were pending for its sale, the figures being $150 000 asked and $100 000 offered. When the water began to come into the sink in large quantities, negotiations were dropped, and the entire plant was soon buried by the Salton Sea.

Operations of the California Development Company.

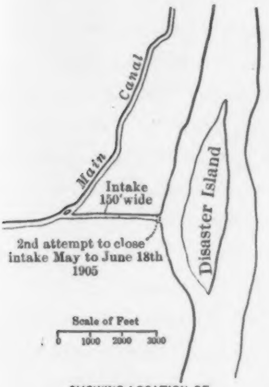
When the C. D. Co. was ready to begin operations, there was on the lower river a dipper dredge with a 4-yd. bucket which had been built and equipped by Hon. Eugene S. Ives, of Yuma, Ariz., and his associates, for digging irrigating canals near Yuma. This dredge was bought by the company in exchange for guaranteed bonds, floated down the river, and, in August, 1900, set to work excavating a canal along the lines marked "Original Intake" in Fig. 9 and then following the old Alamo overflow channel to a point 8 miles below. From that point the Alamo channel, with a little diking here and there, had sufficient capacity to carry for some time the water needed.

About 500 ft. above the Boundary Line a temporary wooden headgate, Fig. 2, Plate CIV, known locally as the "Chaffey" gate, was put

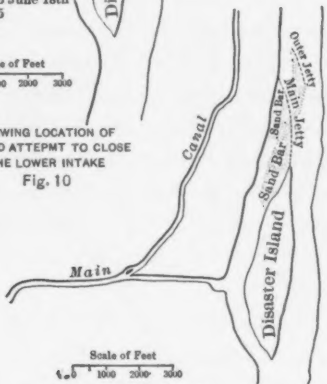
* The published analyses of the deposit give the following average:

Sodium chloride	96.15
Sodium sulphate	0.70
Calcium sulphate	0.60
Magnesium sulphate	1.60
Insoluble	0.10
Water	0.85
	100.00

The California State Mineralogist reports the value as $1 per ton.

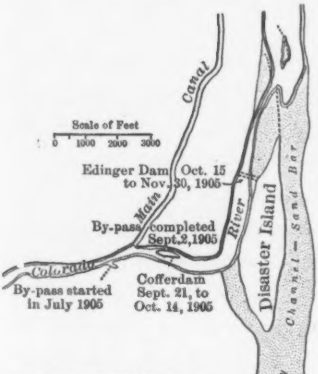


SHOWING LOCATION OF SECOND ATTEPMT TO CLOSE THE LOWER INTAKE
Fig. 10



SHOWING THIRD ATTEMPT TO CLOSE THE LOWER INTAKE JULY, 1905
Fig. 11

MAP SHOWING VARIOUS INTAKES OF THE CANAL TO IMPERIAL VALLEY AND LOCATION OF THE FIRST ATTEMPT TO CLOSE THE LOWER ONE
Fig. 9



SHOWING LOCATION OF WORK ON A HEADGATE AND THE FOURTH ATTEMPT TO CLOSE BREAK
Fig. 12

in. This was a well-designed and well-built wooden, A-frame, flash-board gate, 70 ft. long, 15 ft. high, with a plank floor, and founded on piling. When it was built* nothing was known or even suspected with reference to the rapid and large variation in elevation of the river bed at varying flood stages and otherwise, and it is not surprising, therefore, that the floor was not put as low as it should have been, but, even so, it was not as deep by 5 ft. as planned by Mr. Rockwood, who, by the way, from April, 1900, to February, 1902, was not in charge of the engineering side of the enterprise, Messrs. George and Andrew Chaffey, now of Los Angeles, handling the property. The structure was made no larger, not because of cost, but because it seemed certain that when more water than the gate's capacity should be required, that fact would mean such revenues as to permit building the permanent concrete and steel diversion works at Pilot Knob, regardless of all other considerations. In passing, it may be said that the construction and design of this temporary head-gate was fully equal to that of any throughout the West, even to-day. The floor, however, was quite too high.

At what is known as Sharp's Heading, the Alamo channel was abandoned as the main canal, and the controlling works for the valley end were put in. These consisted of a wooden, A-frame, flash-board gate in the continuation of the Alamo, a similar gate at the head of the Encina or West Side Main canal, and a combined gate and drop, known as Sharp's Head-gate, from which leads off the Central Main, the chief canal in the valley.

This last structure is well worth describing in some detail. In the first place, it is a most vital part of the system, because, being a combination of a drop and regulating gate, were it to fail, the water in the Alamo or Main Canal above it would immediately be lowered far too much to permit taking out any whatever for the East and West Side Mains. To realize the consequences of this, it must be remembered that irrigation water is needed every day in the year, and that no stock and domestic water for the entire region, except for the Town of Holtville, can be had, except from the irrigation system and by being brought in by the railroad in water cars. In the second place, for several months consecutively, in each year since 1905, it has

* Nothing really was known about the changes in elevation of the river bed until 1907.

SHA

All floor and wa

All piles, joists,

Posts and girts

All sheet-piling

Joists rest on c

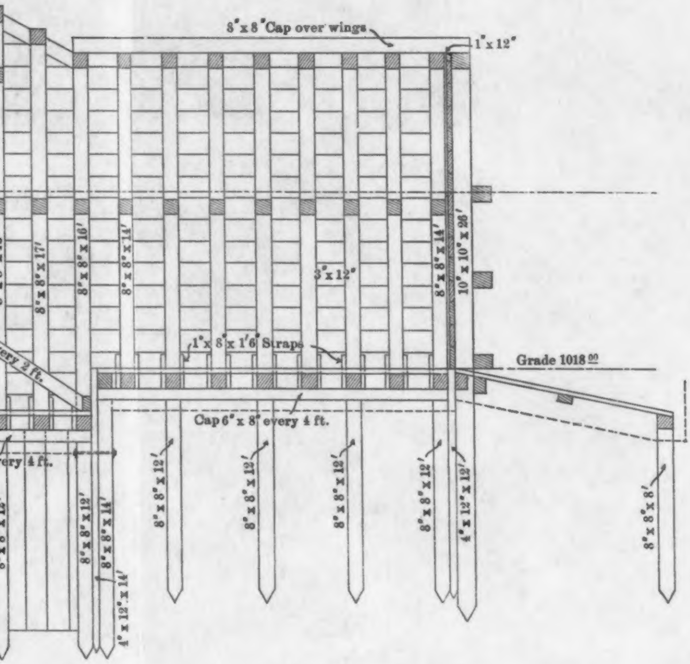
Brush wings ar

SIDE ELEVATION

PLATE CV.
PAPERS, AM. SOC. C. E.
NOVEMBER, 1912.
CORY ON
IRRIGATION AND RIVER CONTROL,
COLORADO RIVER DELTA.

SHARP'S HEAD-GATE IN CENTRAL MAIN CANAL

oor and wall planks are 3"x 12" battened with 1"x 4"
iles, joists, caps, and spreaders, 8"x 8"
s and girts of wings, 8"x 10"
heet-piling has penetration of 11 feet. Double lines represent sheet-piling
s rest on caps which in turn are on 8"x 8" piles
h wings are supported by 8"x 8" piles and girts and lap over wooden aprons.



been taxed beyond the capacity for which it was designed, without developing any serious weakness. Furthermore, at intervals of about 18 months, since it was put in service in 1903, the canal above it has been emptied for periods of not more than 60 hours to permit of inspection and light repairs, but the very first overhauling or extensive repairs were begun on January 5th, 1912.

The writer confesses to a predilection for permanent structures of masonry, concrete, or steel, and this gate and the Alamo Waste-gate, built in 1905, were nightmares to him while in charge of the properties. It would seem that a large part of this was worry wasted, however.

Sharp's Head-gate was designed by, and built under the direction of, Mr. C. N. Perry, then Resident Engineer of both companies, the fundamental idea being to cut up the foundation into a number of water-tight compartments. Plates CV and CVI show this construction.

Where Beltran's Slough leaves the Alamo channel, a wooden, flashboard gate was built to waste water through Beltran's and Garza's Sloughs into the New River, but about 3 months after being put in service it failed, due to back currents below it.

The original plan for supplying the territory to the east of the Alamo was to utilize the Alamo channel from Sharp's Heading to Holtville, an earthen dam being used to bring the water to the surface of the land at that point. This dam soon failed, and the canal from there was connected with the Alamo at a point about 1½ miles above Sharp's, such connection being made in record time, with a cross-section only large enough for the demand. The idea was that erosion would enlarge it, which in general has been the case, although some blasting was required to assist the action. Originally known as No. 5 Main, the canal is generally called the East Side Main. It, as well as the West Side Main, is occasionally broken in places by the severe rainstorms—almost cloudbursts—which occur at infrequent intervals in the region. To provide absolute protection against such damage would be very expensive, and neither No. 5, which owns the exposed portion of the East Side Main, nor the C. D. Co., which owns all the West Side Main, has done so. Otherwise they, as well as the Central Main, are quite satisfactory.

Main canals were constructed from Sharp's to serve the territory between the New and Alamo Rivers (the Central Main); a second, the

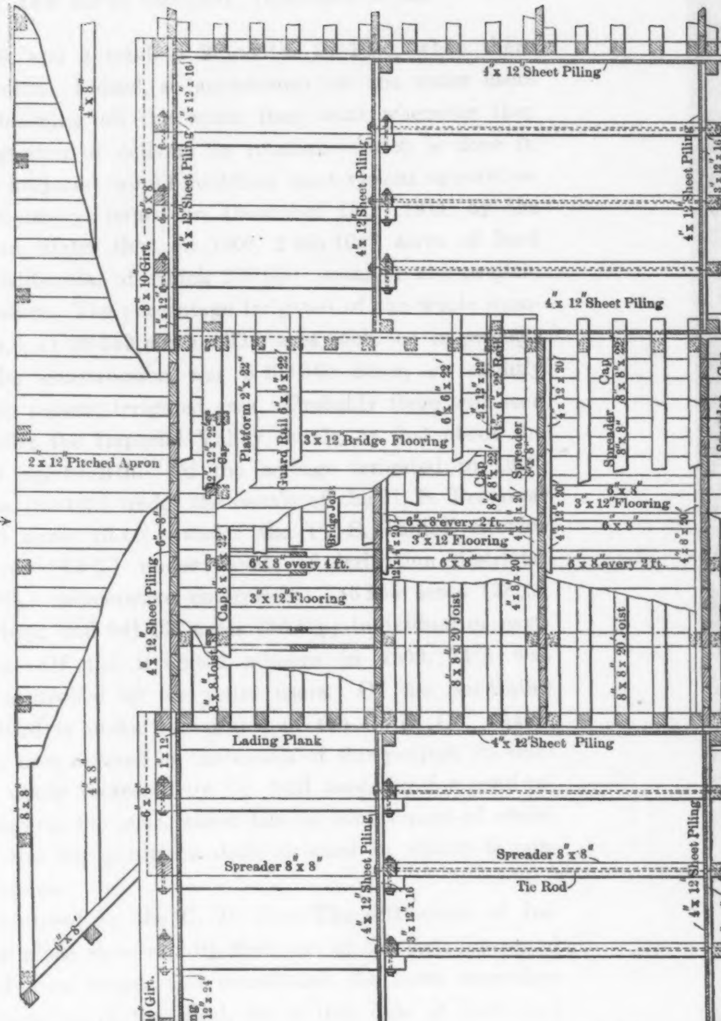
West Side Main, crossed New River to serve territory west of that waterway, and a third, the East Side Main, to serve the territory east of the Alamo. In 11 months, or in June, 1909, delivery of water was begun through the Boundary Canal as far west as Calexico, and the Central Main was put into service in March, 1902, or in 19 months.

Imperial Water Companies Nos. 1, 4, 5, 6, 7, and 8 were formed, and triparty contracts were entered into with each. The C. D. Co. constructed the distributing systems for these districts, with the exception of that of Imperial Water Co. No. 7.* The total length of canals in all these distributing systems was approximately 700 miles on January 1st, 1905, and there were also about 80 miles of canal belonging to the C. D. Co. and the Mexican Co., making the total about 780 miles. During 1905 and 1906 relatively little canal building was done, because the river got beyond control; and, from 1907 to 1911, inclusive, the increase has been less than 20% on account of excessive litigation following the vast expenditures for controlling the river, and because the canals existing on January 1st, 1905, covered 85% of the territory now under ditches.

With the exception of a permanent diversion gate at the river, two permanent structures replacing temporary ones in the valley, the building of the Alamo Waste-gate (Fig. 1, Plate CXIV), just above Sharp's Heading (June 25th to August 17th, 1905), and another in the Central Main at Station 134 (November 13th, 1904, to January 12th, 1905), that portion of the canal system completed on January 1st, 1905, has not been essentially changed or enlarged, and, with few exceptions, the original structures are still being used. There is a marked tendency on the part of the mutual water companies to replace wooden structures with permanent ones of reinforced concrete, but otherwise in general the canal systems are as satisfactory as any which could be devised.

The irrigation service afforded to farmers in Imperial Valley is the best of which the writer has ever heard. This has been the case with the exception of three short periods: the winter of 1904, 1 month

* The water rights for all the land south and east of the district of Imperial Water Co. No. 5 which could be irrigated by gravity from what was known as the Holt Heading—where the East Side Main heads—approximately 18 000 acres, were sold for $50 000, the purchaser, Mr. W. F. Holt, from Mutual Water Company No. 7, constructing the distribution system and selling for his own benefit all the capital stock of this company. The fact that this deal was made at the rate of $3 per acre, including the consideration for the proportional cost of the controlling works in the valley and of the main canal thereto from the Colorado River, for one of the very richest sections of land, shows plainly the financial straits of the company at that time.



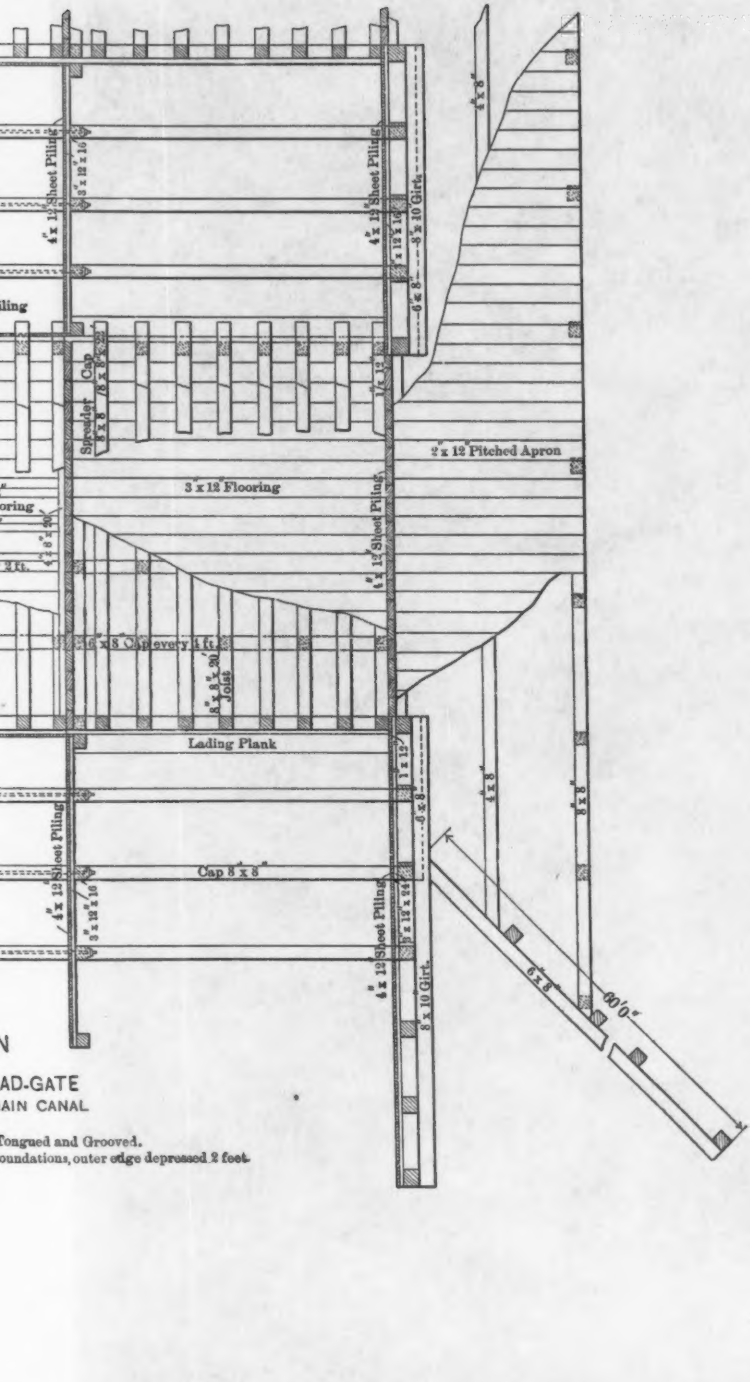
PLAN

SHARP'S HEAD-GATE

IN CENTRAL MAIN CANAL

Double lines represent sheet-piling 4" x 12". Tongued and
Aprons are 2" x 12" flooring resting on pile foundations, ou

PLATE CVI.
PAPERS, AM. SOC. C. E.
NOVEMBER, 1912.
CORY ON
IRRIGATION AND RIVER CONTROL,
COLORADO RIVER DELTA.



N

AD-GATE
AIN CANAL

Tongued and Grooved.
oundations, outer edge depressed 2 feet.

(November) in 1906, and a total of 2 months in 1910, when there were shortages of water. Indeed, so accustomed are the water users of this region to obtaining all the water they want whenever they want it, that a suggestion of delivery in rotation—which is done in almost all irrigation projects—would doubtless meet violent opposition.

A preliminary summary, issued on December 15th, 1911, by the U. S. Census Bureau, states that, in 1909, 2 664 104* acres of land were irrigated in California, of which 220 000 acres, or one-twelfth, were in Imperial Valley. The percentage irrigated of the whole number of farms was 44.6, or 39 352 acres. The area included in projects completed and under construction was 5 490 360 acres, or slightly more than double the present irrigated area. Probably there will soon be 450 000 acres under the Imperial Valley canals, or just about the same proportion of one-twelfth. Of the acreage irrigated in 1909, there were 400 acres (0.01%) under the canals of the U. S. Reclamation Service; 3 490 acres (0.1%) under the U. S. Indian Service canals; 173 793 acres (6.5%) under canals of irrigation districts; 779 020 acres (29.2%) co-operative enterprises; 746 265 acres (28%) commercial enterprises; and 961 136 acres (36.1%) individual or partnership enterprises. Of the irrigated acreage in 1909, 71% was watered by works controlled by the water users. Of the remaining 29%, almost one-third is under the canals of the C. D. Co. Aside from the very large area covered by the canals of this project, its relative importance is vastly increased by the vital necessity for continuous service every day in the year, which has no counterpart of which the writer knows, and the minimum daily demand in winter is one-fourth of the maximum.

Obstacles Encountered by the C. D. Co.—The settlement of Imperial Valley† took place more rapidly than any of the men interested in the project had even hoped, and constituted the most marvelous achievement of irrigation in the West, up to that date at least, and probably to the present time. On January 1st, 1901, with the excep-

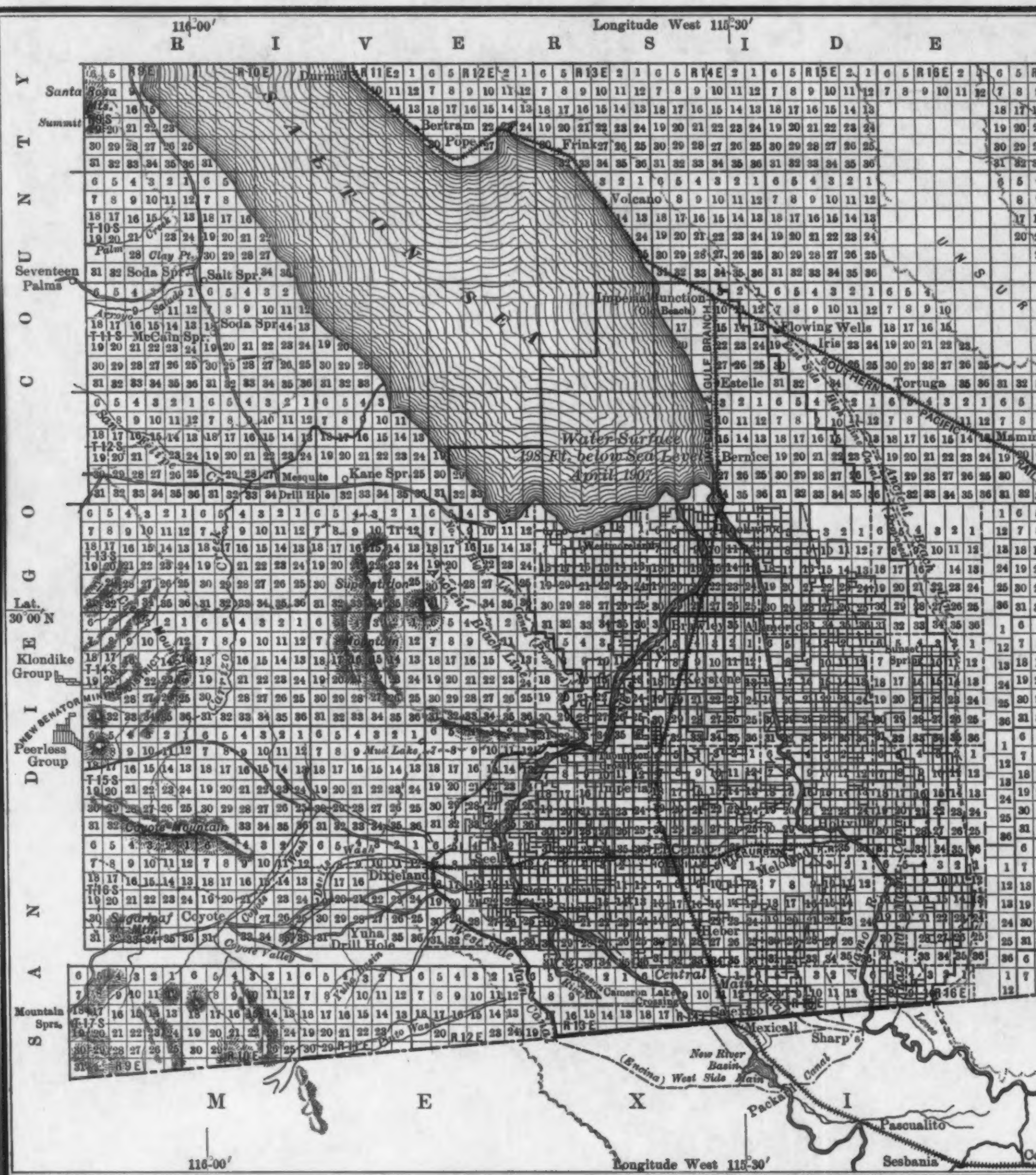
* Undoubtedly, the greater part of this total is irrigated only after a fashion, so that the relative importance of the irrigated area in Imperial Valley is much greater than the figures indicate.

† The Imperial Land Company decided to use the name "Imperial Valley," for the region to be covered by the irrigation canals, instead of "Colorado Desert" or "Salton Basin," partly to distinguish between the reclaimed and unreclaimed areas, but chiefly for the effect of the name on readers of the colonization literature put out by the company. The name, "Imperial Valley," is firmly established as referring to the cultivated portion of the Colorado Delta west of the river, whether north or south of the International Boundary Line.

tion of a party of surveyors, not a single white man lived in the whole region; by January 1st, 1903, 2 000 people had come in; by January 1st, 1904, probably 7 000 people had made their homes in the new district, and by January 1st, 1905, the population was between 12 000 and 14 000. As early as 1904 there was a branch railroad through the district from the Southern Pacific main line at Old Beach, since called Imperial Junction, and, at the beginning of 1905, there were seven towns, with stores, banks, etc., 780 miles of canals, about 120 000 acres of land under actual cultivation, and 200 000 acres covered by water stock.

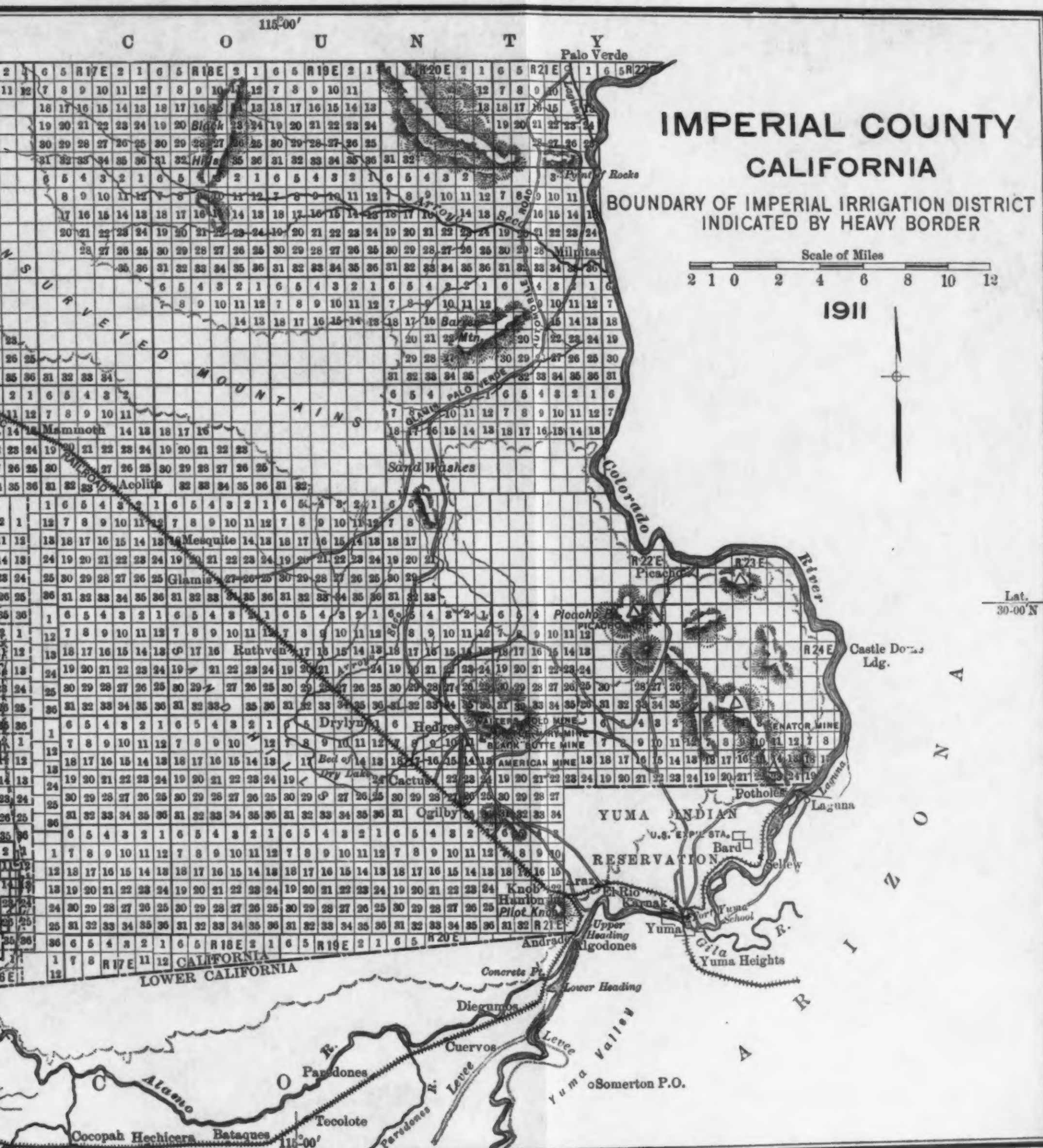
This unprecedented and unexpectedly rapid development overtaxed the resources of the C. D. Co., and, in addition, there were several untoward factors which accentuated the difficulty. These were serious complications in the United States Government Land Survey of the region, an extremely unfavorable soil report by the United States Agricultural Department, agitation for the United States Reclamation Service to supplant the irrigation system of the valley, a question as to the right to divert water from the Colorado River, and troubles at the intake by silt depositions.

United States Land Surveys in the Imperial Valley.—That portion of the Imperial Valley north of the 4th standard parallel was supposed to have been surveyed in 1854-56. The maps and notes for it were accepted, but there is at least some question whether the survey was ever actually made in the field. Later, in 1880, after the International Boundary Commission had surveyed the Boundary Line between the United States and Mexico and marked it continuously with permanent monuments, the area south of the 4th standard parallel was surveyed, this being locally known as the "Brunt" survey. The colonization company, in April, 1900, put out surveying parties under the direction of Mr. Perry, now County Surveyor of Imperial County, to re-run the Government lines and establish corners so that settlers might have proper descriptions for the tracts of land they wished to file on, and also that the distribution systems of the various mutual companies might be located along the Government subdivision lines, as the topography of the land is such that this ideal canal location is generally feasible. Mr. Perry found nearly all the corners of the Brunt survey, and used the notes showing certain connections made with the survey of 1856 along the 4th standard parallel. In this way



116°00'
Longitude West 115°30'
R I V E R S I D E
SAN DIEGO COUNTY
R 9 E
R 10 E
R 11 E
R 12 E
R 13 E
R 14 E
R 15 E
R 16 E
T-10-S
T-11-S
T-12-S
T-13-S
T-14-S
T-15-S
T-16-S
T-17-S
Santa Rosa Mts.
Summit
Durmid
Bertram
Pope
Frink
Volcano
SALTON SEA
Seventeen Palms
Palm
Clay Pt.
Soda Spr.
Salt Spr.
McCain Spr.
Soda Spr.
Arroyo Salado
Mesquite Drill Hole
Kane Spr.
Imperial Junction (Old Beach)
IMPERIAL & GULF BRANCH
Flowing Wells
Iris
SOUTHERN PACIFIC
Tortuga
Estelle
Bernice
Water Surface 198 Ft. below Sea Level April 1907
Lat. 30°00' N
Klondike Group
NEW SENATOR MINE
Peerless Group
Superstition Mountain
Mud Lake
Coyote Mountain
Dixieland
Seeley
Imperial
Brawley
Keystone
Holtville
El Centro
Heber
Calexico
Mexicali
Sharp's
Central
Cameron Lake Crossing
Sugarloaf Mtn.
Coyote
Coyote Valley
Yuha Drill Hole
West Side Main Canal
Mountain Sprs.
New River Basin
(Encina) West Side Main
Packard
Pascualito
Sesbania
M E X I
116°00'
Longitude West 115°30'

PLATE CVII.
PAPERS, AM. SOC. C. E.
NOVEMBER, 1912.
CORY ON
IRRIGATION AND RIVER CONTROL,
COLORADO RIVER DELTA.



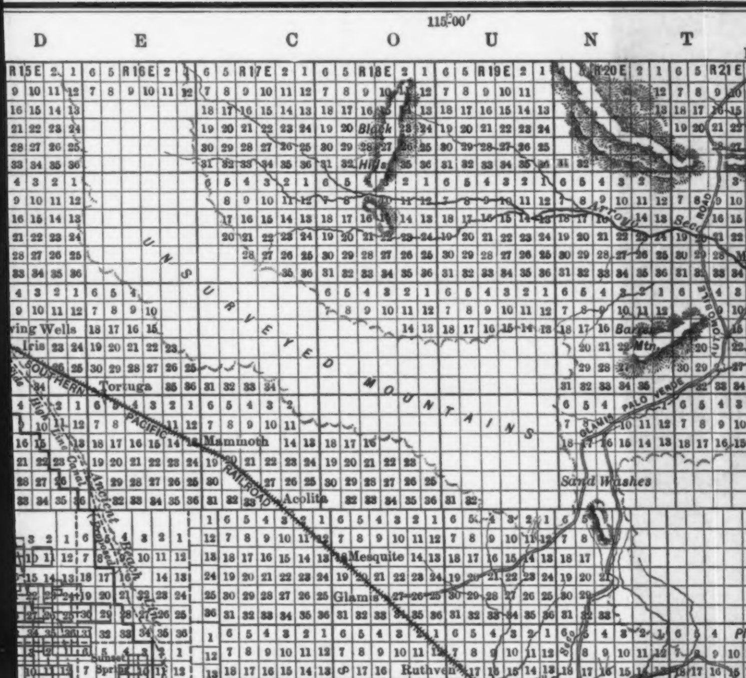

115°00′
D E C O U N T
R15E
R16E
R17E
R18E
R19E
R20E
R21E
Black Hills
UNSURVEYED MOUNTAINS
Barren Mtn.
Sand Washes
Iris
Tortuga
SOUTHERN PACIFIC RAILROAD
Mammoth
Acolita
Mesquite
Glamis
Ruthven
Drylyn
Hedges
Bed of Dry Lake
Cactus
Ogilby
Knob
Hanlon
Pilot Knob
Sunset Springs
CALIFORNIA
LOWER CALIFORNIA
Andrade
Concrete Pt.
Cuervos
Paredones
Paredones R.
Alamo R.
Levee
Tecolote
Pascualito
Sesbania
Cocopah
Hechicera
Bataques
I C O
115°00′

PLATE CVII.
PAPERS, AM. SOC. C. E.
NOVEMBER, 1912.
CORY ON
IRRIGATION AND RIVER CONTROL,
COLORADO RIVER DELTA.

the lines to the north were retraced, but, some time later, when the survey had extended farther and the work of retracing the lines east of the Alamo River was commenced, it was discovered, by encountering natural features given by the notes of the 1856 survey, 2 miles or more out of correspondence, that there were serious errors. Exhaustive search was then made for the 1854-56 survey stakes, but in an area of thirty townships only five corners were discovered which seemed to be authentic. These were widely scattered, and showed great errors. Between the 3d and 4th parallels the actual distance was found to be 25½ miles, or an error of 1½ miles in a 24-mile north and south line. East and west the error was approximately 2 miles in 30.

Throughout the territory, Sections 16 and 36, the school sections, had been given to the State of California by the United States Government for the benefit of the State school fund, the remainder of the land belonged to the United States, and this dual ownership increased the difficulty of making any adjustment. In June, 1902, the president of the colonization company and the chief engineer of the C. D. Co. went to Washington, explained the situation, and, on the advice of the General Land Office, an Act was prepared and passed in July, 1902, authorizing a resurvey of twenty townships of the land in Imperial Valley. The outside lines of these townships were re-run in 1903 and are known locally as the "Henderson" survey. It was more than 6 years, however, before the interior lines in these townships were re-run and the work was completed and approved.

In the mean time, it was impossible for the Land Office to issue patents to the settlers, and thus men practically owning from 160 acres to two and three times that area of extraordinarily fertile land, with a selling value of from $60 to $100 per acre, could offer no security for a loan with which to make permanent improvements. The United States land laws are extremely strict and severe with reference to a settler borrowing money with which to make final proof. Under such circumstances, the interest rate was naturally from 10 to 12% per annum, while the interest on deferred payments for the water stock was only 6%, so that the C. D. Co. suffered severely. However, it was not until 1909, more than 3 years after the control of the company was taken over by the Southern Pacific interests, that any suits were entered to foreclose on the collateral notes and mortgages secured by the water stock.

Soil Surveys of the Imperial Valley.—In the fall of 1901 the Bureau of Soils, United States Department of Agriculture, made a soil survey of Imperial Valley. On January 10th, 1902, a preliminary report, "Circular No. 9," was issued covering the 169 sq. miles of territory which had been examined.* The report doubtless presented the only possible conclusions, according to the information at that time extant regarding alkaline soils of such depth as are found in the Imperial Valley. It was very unfavorable, however, and calculated to deter sensible people from settling in the region. For example, one statement was as follows:

"One hundred and twenty-five thousand acres of land have already been taken up by prospective settlers, many of whom talk of planting crops which it will be absolutely impossible to grow. They must early find that it is useless to attempt their growth. * * * No doubt the best thing to do is to raise crops such as the sugar beet, sorgum, and date palm (if the climate will permit), that are suited to such alkaline conditions, and abandon as worthless the land which contains too much alkali to grow those crops."

The warning was reiterated in a subsequent report.† It seems certain that, had the territory not been already settled in very large measure when these reports were sent out, Imperial Valley would yet be unreclaimed.

Agitation in Favor of a Reclamation Service Project.—When the United States Reclamation Service Act was passed, in June, 1902, the crops produced in the Imperial Valley were causing a return of confidence in the region, and the extraordinarily rapid development was being resumed. The irrigation possibilities on the Colorado River had already been examined by the United States Geological Survey, and in 1903 plans for the Yuma Project were outlined. The engineers of the Service were convinced that no diversion from the Colorado for irrigation could be permanently successful where provisions were not made for preventing the heavy silt from entering the canals—that it would take an impractically large amount of dredging to keep canals leading directly from the river open for reasonably satisfactory delivery of water. The cost of the Laguna Weir, borne by the land owners of Imperial Valley alone, constitutes a serious burden, but, if

* "Field Operations of the Bureau of Soils, U. S. Department of Agriculture, 1901, p. 587."

† "Soil Survey of the Imperial Area, California (Extending the Survey of 1901), Advance Sheet of Field Operations of the Bureau of Soils, 1903."

borne by all the irrigible land in both valleys, the cost per acre would be reduced to approximately one-fourth. Mr. William E. Smythe, of San Diego, who has been very prominent in the work of the National Irrigation Congress, and has written extensively on irrigation generally, urged the people of Imperial Valley to join with the Yuma Project; that the enterprise would then be backed by the Government with unlimited funds; that they would be required to pay to the Government only a small portion of the money they were obliged in one way and another to pay the C. D. Co., and that they would eventually acquire the laudable desire of owning and operating their own system. The Imperial Water Users' Association was accordingly formed with Mr. W. F. Holt, of Redlands, Cal., as its President, and negotiations were at once instituted with the C. D. Co. to acquire its canal system. Mr. A. H. Heber, President of the C. D. Co., who acted for it in the matter, knew that the estimates of the Reclamation Service for the canal line into Imperial Valley, lying entirely on American soil, were at least $10 000 000, on account of the sand hills. He believed that the Alamo River for 40 miles was a very satisfactory main canal, and that by owning the 100 000-acre tract of the Mexican Co., building another waterway through Mexican territory would require the consent of the Mexican Co.; consequently, his idea regarding the values of the property were excessively high.

As a natural feature of these negotiations, and with a view to tempering such ideas as to price, the right of the C. D. Co. to take water from the Colorado was challenged. The navigability of that stream suddenly assumed serious commercial, national, and international importance. As usual in such cases, these questionings were carried to unfortunate extremes.

In the course of events, at a mass meeting of the farmers in Imperial on July 30th, 1904, Mr. Heber offered to have the price fixed by arbitration, one man to be appointed by him, one by the Imperial Water Users' Association, and a third to be selected by these two. This was not done, but instead, the engineers of the Reclamation Service estimated the value of the plant of the C. D. Co. and the Mexican Co., making a report to the Secretary of the Interior on October 1st, 1904, a copy of which the writer has not yet succeeded in obtaining. On being advised by the Secretary of the Interior of the conclusion of such report, the Imperial Water Users' Association ap-

pointed a committee, headed by Mr. Holt, to negotiate with Mr. Heber, which was done, and a price of $3 000 000 was mutually agreed on. A petition was addressed to the Secretary of the Interior setting forth such action, and the committee of the Water Users' Association, together with Mr. Heber, as the duly authorized agent of the C. D. Co., went together to Washington to arrange matters accordingly. Soon after reaching Washington, however, the committee, without intimating to Mr. Heber in any way that it had changed its opinion, agreed with the Reclamation Service authorities against buying the property on such a basis.

With such unpardonable bad faith on the part of the committee, it is not surprising that the conference ended with relations between Mr. Heber and the Reclamation Service so strained that further negotiations were impossible. At that time it was announced by the Service that its legal department had concluded that no law existed whereby it could deal with the problem of carrying water through Mexico.

The effect of the entire incident was to render the people of the valley antagonistic to the company, and at the same time split them into several factions. More important, however, was the effect of the severe criticisms of the plant and water rights of the C. D. Co., which had been given wide publicity. The company's credit, which had slowly but steadily improved since 1902, was again destroyed in Southern California and in the larger financial markets of the United States. Consequently, early in 1905, when these negotiations ended, the company was almost on the rocks.

Water Rights Attacked.—Because of the attacks on the right to take water from the Colorado, then well under way, a bill was introduced into the House of Representatives in January, 1904, at the request of the C. D. Co., declaring:

"That the water of the Colorado River for the irrigation of the arid land that may be irrigated therefrom is hereby declared to be of greater public use and benefit than for navigation, and the diversion of the water from said river, heretofore made and that which may in future be made, for irrigation purposes, in accordance with the laws of the respective States and Territories in which such diversion has been or may be made, is hereby legalized and made lawful.

"Section 2. That any person, firm, or corporation be, and is hereby, authorized to divert, take, and appropriate water from the Colorado River for the purpose of irrigation, in such quantity, subject to and

under the State appropriation of the State of California, as now in force under the laws of said State." (H. R. 13 627, 58th Congress, 2d Session.)

The U. S. Reclamation Service had filed on some of the flood-waters of the Colorado in order to fill four large reservoirs between The Needles and Yuma, then under contemplation, and such filings were practically second only to those of the C. D. Co., so that the effect of this proposed legislation, other than on the C. D. Co., was null. The bill was bitterly opposed by Mr. Smythe, as representing the majority of the settlers in Imperial Valley. No attempt was made to amend the bill with a view of protecting all interests in a fair and equitable manner, but instead, under date of April 8th, 1904, the Acting Attorney General, Mr. Hoyt, in an opinion addressed to the Committee on Irrigation on Arid Lands, to which the bill had been referred, said:

"In view of these provisions [from the Treaty of Guadalupe-Hidalgo, February 2d, 1848; of the Gadsden Purchase, December 30th, 1853; and the Boundary Treaty of November 12th, 1884, between Mexico and the United States] and of the important irrigation projects now and hereafter to be carried on by the United States Government, I seriously doubt the wisdom of a surrender by Congress at this time of all control over the waters of the Colorado River."

Accordingly, the Committee reported* requesting the Secretary of the Interior to investigate and report to Congress on the various questions involved in the use of the waters of the Lower Colorado River, with a view to determining their availability for irrigation, and recommend any legislation which might be necessary. This resolution failed to pass.

Mexican Concession Secured.—Failing to secure an adjustment of water rights at the hands of Congress, Mr. Heber went at once to Mexico and quickly obtained a concession from President Diaz, which was ratified by the Mexican Congress on June 7th, 1904.

This concession authorized the Mexican Co. to carry, through its canal system in Mexico, 284 cu. m. per sec. (approximately 10 000 sec-ft.), to be diverted from the Colorado River in United States territory by the C. D. Co. and turned over to the Mexican Co. at the boundary line; to construct an intake on Mexican territory, and connecting with the said canal system, and divert through such intake

* House Joint Resolution No. 147.

284 cu. m. per sec., to be used in the irrigation of lands in Mexico and in the United States, but with the proviso, "without injuring the rights of any third party nor the navigation as long as the river is destined for navigation"; that, of the water carried in the canal, enough should be used to irrigate the lands in Mexico susceptible of irrigation by gravity to an amount not exceeding one-half the total volume; that the Mexican Co. should begin surveys within 6 months, and within 12 months file, with the Secretary of Development, maps in duplicate of the proposed extensions and betterments, together with a descriptive report, and entirely complete the same within 7 years; that the company should pay into the Inspection Fund, as is customary in all concessions granted by the Mexican Government, a sum, in this case $300 (Mexican money), per month, and should be subject to inspection by an engineer appointed by the Secretary of Development; granting the company the right of eminent domain over private property and defining the process by which condemnation could be carried out—incidentally with minimum possible difficulty—and permitting importation once for all, free of customs or duty, all equipment and apparatus necessary for the construction of the proposed extensions and betterments, together with freedom from all taxes, except stamp tax, for a period of 10 years; stipulating that under no circumstances should the company sell or mortgage the concession to any government or foreign state, nor admit it in partnership; that the company should be subject to the laws and rulings now in force, and which in future may be enacted, for the supervision and use of waters; particularly specifying that the company and its assigns shall always be considered as Mexican corporations, though all or any of its stockholders should be foreigners; that the corporation should be subject to the jurisdiction of the Courts of the Republic in all affairs emanating within Mexican territory, and that such stockholders should never be able to allege the rights of foreigners under any circumstances, but have the rights and the methods of establishing the same as the laws of the Republic grant to Mexican citizens, so that, in any matters, diplomatic or foreign agents should not have any interference.

Condition of Plant in the Summer of 1904.

From the first, there was a great deal of trouble with the Chaffey Head-gate, chiefly because its floor was not down to the bottom grade

line of the canal, as originally planned. As has been explained, this gate was a temporary structure, but well and substantially built. Just as it was being covered up by the operations of the dredge, *Alpha*, cutting the main canal into the permanent concrete head-gate from below, in 1906, the writer examined it carefully and found it in an excellent state of preservation. The floor was so high, however, that it was necessary, during the low-water seasons of 1902-03 and 1903-04, to cut a by-pass around the gate, and close it on the approach of the summer floods. When the Mexican concession was obtained, the first Mexican intake was cut from the river to the main canal, as shown in Fig. 9.

In the winter of 1902-03 there had been shortages of water in the valley, due to the fact that the main canal had not been completed to its final depth; and, with the apparatus and available funds on hand, it was impossible to keep the water supply up to the demands when the river fell exceedingly low. In the winter and early spring of 1904, another water shortage caused considerable damage in the valley, and claims amounting to $500 000 were presented to the company. Every one of these was settled out of Court, however, in 1905 and the early part of 1906, with a payment of less than $35 000, taken entirely in water and water stock, and the writer believes that every claim was fairly settled, at least as far as the settlers were concerned.

Below the intake the first 4 miles of the Main Canal caused much worry, due to the extent to which it silted up during floods, but, with this exception, the plant of the C. D. Co. was in quite satisfactory condition. The canals were generally well located and in fair condition, and the structures, while of redwood and not concrete, were substantially built according to good design, and were in excellent condition. The canals in the distribution systems of the mutual water companies were silting up constantly, on account of the muddy water. In part, this was unavoidable, but was largely due to uneconomical methods of water deliveries when dealing with muddy water, particularly in serving any settler on his demand, regardless of the very low velocity, if no one else wanted water from the lateral during the same time. The silt problem in the distribution systems of these companies, however, is as simple as it will ever be for any lands irrigated along the Lower Colorado. The financial status of the various mutual com-

panies was quite good, and they had generally established a small but satisfactory credit with the local banks.

To avoid excessive silt depositions in the first 4 miles of the canal, In February and March, 1904, the Best Waste-gate, so-called, was put in 8 miles below the intake, where water could be wasted from the Alamo channel through the Quail River into the Paredones River and thence into Volcano Lake. This was a wooden A-frame, flash-board gate, 60 ft. long, but it was carried away in June, 1906, by the side-cutting of the banks while the Alamo channel was being enlarged by that year's summer flood. The idea was to divert a large quantity of water during the flood season of 1904, waste it through the Best Gate, and in this way scour out the upper portion of the canal. At first the action was as expected, and some 2 ft. in the bottom were carried away. When the river reached its maximum height during the summer flood of 1904, however, and carried an excessive silt content, particularly of the heavier and sandy type, this scouring action was entirely overcome, and the bottom of this stretch was raised approximately 1 ft. higher than during the previous year.

The Silt Problem.—This action accentuated, and properly impressed on the engineers of the C. D. Co., the seriousness of the silt problem in diverting the Colorado River water. Generally speaking, during flood stages, the water carries all the silt it can transport, and the faster the current the larger the particles it picks up and carries along. It is certainly desirable, and probably essential, to provide settling basins at or immediately below the diversion point, in which water can be practically stilled and thus insure the deposition of the heavier silt having very slight fertilizing value, and the admission of only such partly clarified water into the canals. Unless some provision is made, as at the Laguna Weir, the diversion canal immediately below the head-gate must act as a settling basin, which is just what happened from the very beginning in the canals of the C. D. Co.

The results of such excessive silting were obviated in various ways, largely by the construction of new intakes, until the diversion of the entire river occurred, and the permanent head-gate was put in service in 1907. The clam-shell dredge, *Delta,* was utilized intermittently to remove the deposits until 1910, then a submerged weir was built across the river, to raise the water at the intake; and lastly large suction dredges were operated just below and just above the regulating gates.

Rather carefully kept records indicated that the bed of the canal at the Lower Heading was raised a little more than 5 ft. between March 1st, 1907, and March 1st, 1910, most of this taking place in the first 6 months. The bed of the Alamo near Sharp's Heading was raised approximately 2 ft., in the same time, and there is constant deterioration all along between these points on the Alamo channel. The reduction of capacity in the larger canals has been noteworthy, but the maximum effect is shown in the smaller laterals constituting the distribution systems of the various mutual water companies.

Mr. Robert G. Kennedy states* that on the Bari Doab Canal from the Punjab River, the canals in Sind from the Indus and Shwebo, and the Mandalay canals in Burmah, it appears that in a non-silting and non-scouring channel the mean velocity is independent of the width, but increases with the depth of the channel, according to the equation:

$$V_o = 0.84\, d^{0.64},$$

in which V_0 = the mean velocity of a non-eroding, non-depositing current; and d = the depth for fine sand-silt, the constants varying slightly with the kind of silt.

He also points out the exceedingly important deduction that during flood stages in the river, the diversion of large quantities of water in an effort to scour away silt depositions in the upper reaches of canals will have the opposite result, because of the excessive silt contents of the water diverted.

The same rule probably applies fully in the case of canals carrying Colorado River water when they are free of vegetation. In point of fact, however, rank growths of tules and willows spring up on the banks and berms and along the edges out into the water with such rapidity as to increase tremendously the deterioration of carrying capacity, particularly in the smaller canals. Furthermore, the rate of deterioration in these laterals increases with the decrease in channel efficiency. The maintenance of the district distribution systems, therefore, consists, in large part, in keeping down and removing the brush and tules.

The various distribution systems were ordinarily designed and built on the basis of a capacity of 1 sec-ft. per 120 acres of land there-

* "The Prevention of Silting in Irrigation Canals," *Minutes of Proceedings*, Inst. C. E., Vol. CXIX, 1895, pp. 281-290.

under, although in some cases the ratio was decreased to 1 sec-ft. per 93 acres (8 in. vertical depth of water in a month). It would have been just as well, indeed considerably better, and of course cheaper, to have made the canals much smaller, for they were put into service when only a small percentage of the land was in cultivation, and, as they carried only a fraction of their capacity, they very soon silted up badly. Removing the silt deposition and the accompanying tule growths is fully as expensive as the excavation of the original section.

This needlessly large excavation was required by the contract provisions under which the C. D. Co. built the distribution systems of the various mutual water companies, and such provisions at the time were necessary to assure colonists that the water supply would be ample. In the construction of the first lateral canals built, however, the leaving of inside berms was a defect which should have been avoided. These flat stretches, usually kept damp and seldom deeply submerged, afford ideal conditions for tule growths, and should be studiously avoided in this region.

Canal Maintenance.—In general, the best method of clearing away the brush tules and deposited silt in the smallest canals has been found to be by Mexican or Indian hand labor. The presence of checks and other canal structures at relatively close intervals makes the use of machinery of questionable economy. For the large canals, "V's", dragged by horses or traction engines, portable floating dipper-dredges, Lidgerwood cross-drags, portable clam-shell dredges, and a number of devices designed by local inventive geniuses have been tried with varying success. The results in all cases depend so greatly on the efficiency with which they are handled and the local conditions under which they work that it will not be profitable to attempt to give any cost figures—indeed, with the exception of Imperial Water Company No. 1, no cost-keeping worthy of the name has been attempted.

Perhaps the most satisfactory appliance for cleaning canals too small for floating dredges is a clam-shell bucket arranged on wheels so that it may follow along the bank. (Fig. 1, Plate CVIII.) The C. D. Co. has two of these machines, manufactured by the Stockton Iron Works, Stockton, Cal., which cost $5 000 each, f. o. b. factory. These consist of a clam-shell bucket having a capacity of 15 cu. ft., with a 40-ft. steel boom carried on an all-steel frame. The maximum width is 14 ft. The power is supplied by a 15 h.p. Atlas gasoline

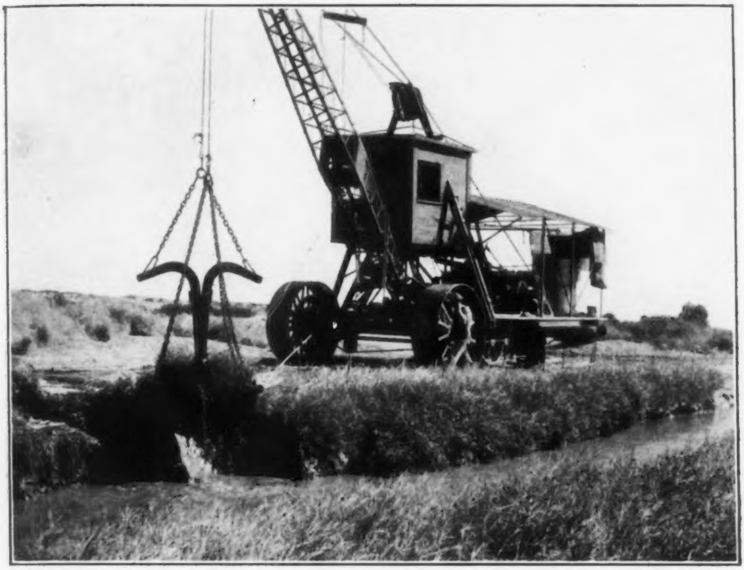


FIG. 1.—STOCKTON CLAM-SHELL DREDGE CLEANING CANALS IN IMPERIAL VALLEY.

FIG. 2.—THIRD ATTEMPT TO CLOSE BREAK AT LOWER MEXICAN INTAKE, JUNE 1ST, 1905.

engine, manufactured in San Francisco, and the machine is self-propelling, with two speeds forward and one reverse. No definite figures, including deterioration and cost of moving from one job to another, are available, but it is understood that the cost of handling material with these machines is about 13 cents per cu. yd.

For handling silt in the upper reaches of the Main Canal along the river, the large 4-yd. dipper dredge, *Alpha*, used in the original construction, was perhaps the most efficient of all agencies for the first few years until the waste banks along the canal became too high to permit of its further use; it handled material for about 6 cents per cu. yd. A suction dredge, the *Beta*, equipped with a 12-in. Kroh centrifugal pump—manufactured in San Francisco—was tried very soon after the canals were put into service, but too much difficulty was caused by roots clogging the pipes and machinery. Mr. H. W. Blaisdell, of Los Angeles, one of the principal stockholders of the C. D. Co., devised a rotary cutter for use at the end of the suction pipe, but it was not successful. This dredge was used at the Lower Heading, in the construction of the Rockwood Gate, and in the subsequent diversion work until June, 1907, when it was dismantled.

In the central main in the valley, and also in the Alamo channel just above Sharp's Heading, a 2-cu. yd. dipper dredge, *Gamma*, has been used almost continuously since it was put in operation in 1904, removing material at about 5 cents per cu. yd.

The clam-shell dredge, *Delta*, described in some detail later, has done excellent service in silt removal and incidental levee building, as well as in channel straightening, since its arrival on the work in November, 1906. It is now engaged in building cut-offs and making general channel improvements, rather than removing the silt deposits direct.

In the summer of 1910 an arrangement was entered into between the various mutual water companies combined and the Receiver of the C. D. Co. whereby the former was to furnish the money and build a suction dredge and rent it to the latter for 10% annually on its first cost. This dredge was built just below the concrete head-gate, and its operation is confined to the American side of the line, the contract being entered into with the North American Dredging Company, of Los Angeles, on December 10th, 1910, for the construction and equipment of an exact duplicate of one of the latter company's dredges in

San Pedro Harbor, for $57 300. After being put into service it was found necessary to remodel the upper deck, in order to make the quarters of the crew suitable for the climatic conditions, at a cost of $950, and a bonus of $2 200 was paid for completion 11 days ahead of the contract time—4 months—making the total cost $60 450, exclusive of engineering, inspection, and legal expenses, which brought the grand total cost up to approximately $63 600. This dredge, the *Imperial,* has a hull 105 ft. long, 55 ft. wide, and 8 ft. deep, and is equipped with a 15 by 60-in. Kroh centrifugal pump driven by a vertical compound engine, steam being supplied by a 250-h.p. marine-type boiler. This dredge handles the silt deposits in the enlarged section of the canal below the head-gate at the rate of about 200 cu. yd. per hour lifted to an average height of 35 ft., at a cost of from 5 to 7 cents per cu. yd., exclusive of interest, taxes, and depreciation, using crude oil fuel at $1.40 per bbl., equivalent to coal at $5.60 per ton.

The *Imperial* was equipped with a cutter for stirring up the material, but this was found to be unnecessary for handling the silt deposits just below the head-gate, and the cutter engine, of vertical compound type, with 8 by 15 by 10-in. cylinder, was installed on the barge, *Silas J. Lewis,* mentioned later, in the canal above the head-gate to run the 10-in. Kroh pump formerly on the *Beta,* the resulting dredge being known as the *El Centro.* Under like conditions, the cost of handling material with the *El Centro* is approximately the same as with the *Imperial.*

With these two suction dredges, it is claimed that the bed of the Main Canal has been lowered approximately 5 ft. above and at the head-gate and for a distance of 3½ miles below, diminishing gradually to nothing throughout the next 2½ miles. If future experience confirms such results, it would seem that the periodic dredging of silt depositions from a settling basin near the intake, at a cost of from $30 000 to $40 000 per annum, will solve the silt problem in the Imperial Valley canal system, except for the very fine silt which cannot be gotten rid of except by allowing the water to be quiescent for some time.

The following general cost figures on maintaining the 373.25 miles of canals of the distribution system of Imperial Water Company No. 1 during 1911 are taken from the annual report of the Superintendent, R. S. Carberry, Assoc. M. Am. Soc. C. E.

Cleaning 465 miles of canal cost $60.64 per mile. The figures in 1910 are 562 miles at $43.81 per mile, and the average cost for the last 6 months of 1909 was $73.16. Clearing on 194 miles of canal cost $35.39 per mile. Cutting brush on 392 miles cost $20.71 per mile. The figures in 1910 were 346 miles at $43.47 per mile, and $60.65 per mile for the last 6 months of 1909.

In this report it is stated that canal "V'ing" is the best method for cleaning canals, generally speaking, and the company owns three "V's", each costing about $600, and three caterpillar traction engines to operate them, each costing $4 200. During the year, 363.8 miles of canal were "V'd" at $58.91 per mile, as compared with 362 miles in 1910 at $60.74 per mile, the details being as follows:

"V'ing"	$16.76
Repairs to engines	16.29
Repairs to "V's"	5.00
Fuel and oil	5.96
Mexican labor following "V's"	14.80
Total average cost per mile	$58.91

During the year, 1 415 miles of canal were worked on, so that the whole system was covered in various ways nearly four times during the year. A small portion of the system was not worked on at all, so that this statement gives some idea of the difficulty in maintaining the system.

The cost of building 117 new structures was $6 278.75, and the cost of repairing old structures was $4 145.05, making the total cost of structure maintenance and renewal $10 423.80. The average number of men employed per day (300 working days per year) was 162, or 0.43 man per mile of canal per day, in addition to teams and machinery. The bottom width of the canals constituting this system varies from 20 to 5 ft.

Canal Operation.—The mutual water companies have never considered delivering water to stockholders in rotation, but instead, without exception, supply any water user on demand, even though he may be at the very end of a long lateral and the only person desiring water from that lateral at the time. Thus, naturally, exceedingly small quantities of water are carried occasionally in every canal except the

very largest laterals, and the result is low velocities and heavy silt deposition and canal deterioration. The feeling seems to be general that the additional cost of maintaining the various distribution systems is more than offset by the advantages or convenience of the water users in obtaining irrigation water at all times on 24 hours' notice. The amount which the maintenance cost of canals could be cut by adopting a rotation system of delivery is problematical, but must be between 35 and 65 per cent. This fact should be borne in mind in making comparisons with the cost data just given.

The Fourth or Lower Intake.

This is such a very important matter that the reasons for digging the lower Mexican intake and the method of handling it when completed are given by quoting from Mr. Rockwood,* the man who did it.

"As soon as the summer flood (1904) dropped and I discovered that instead of the bottom being lower it was approximately 1 ft. above that of the year previous, we adopted the only means at our command to attempt to deepen the channel.

"Knowing the character of the material to be removed, we knew that with the dredging tools which we had (4-yd. dipper dredge *Alpha* and 12-in. suction dredge *Beta*), it would be impossible to dredge out this 4 miles of canal in sufficient time for the uses of the valley, providing the water in the river should drop as low as it had the previous year. The dredges were brought back, however, and put at work, but the result proved, as I had anticipated, that it would take practically all winter to dredge the canals; that is, it would take all winter to provide new machinery, even if we had the money; and in hopes, then, that it might possibly prove effective, I employed the steamer *Cochan,* and, placing a heavy drag behind it, ran it up and down the canal in hopes that by stirring up the bottom there would be sufficient velocity in the canal itself to move the silt deposits on below the 4-mile stretch to a point where I knew the water had sufficient velocity to keep the silt moving. A month's work, however, with the steamer proved that the work being done by it was inadequate.

"*The Great Problem.*—We were confronted then with the proposition of doing one of two things, either cutting a new heading from the canal to the river below the silted 4-mile section of the canal, or else allowing the valley to pass through another winter with an insufficient water supply. The latter proposition we could not face for the reason that the people of the Imperial Valley had an absolute right to demand

* "Born of the Desert—Imperial Valley In Its Making Not A Dream—A Brief History of the California Development Company." By C. R. Rockwood. Second Annual Magazine Edition, *Calexico Chronicle*, Calexico, Cal., May, 1909.

that water should be furnished them, and it was questionable in our minds as to whether we would be able to keep out of bankruptcy if we were to be confronted by another period of shortage in this coming season of 1904-1905.

"The cutting of the lower intake, after mature deliberation, and upon the insistence of several of the leading men of the valley, was decided upon. We hesitated about making this cut, not so much because we believed we were incurring danger of the river's breaking through as from the fact that we had been unable to obtain the consent of the Government of Mexico to make it, and we believed that we were jeopardizing our Mexican rights should the cut be made without the consent of the Government. On a telegraphic communication, however, from our attorney in the City of Mexico, to go ahead and make the cut, we did so under the presumption that he had obtained the necessary permit from the Mexican authorities. It was some time after this, in fact after the cut was made to the river, before we discovered that he had been unable to obtain the formal permit, but had simply obtained the promise of certain officials that we would not be interfered with, providing that plans were at once submitted for the necessary controlling structures to be placed in this heading.

"*Reasons Why.*—This lower intake was constructed, not, as is generally supposed, because there was a greater grade from the river through to the Main Canal at this point. The grade through the cut and the grade of the Main Canal above the cut were approximately the same, but the cut was made at this point for the reason that the Main Canal below the point where the lower intake joined it, was approximately 4 ft. deeper than the Main Canal through the 4 miles above this junction to the Chaffey gate, consequently giving us greater water capacity. In cutting from the Main Canal to the river at this point, we had to dredge a distance of 3 300 ft. only, through easy material to remove, while an attempt to dredge out the Main Canal above would have meant the dredging of 4 miles of very difficult material. We began the cut the latter end of September and completed it in about 3 weeks.

"As soon as the cut was decided upon, elaborate plans for a controlling gate were immediately started and, when completed early in November, were immediately forwarded to the City of Mexico for the approval of the engineers of the Mexican Government, without whose approval we had no authority or right to construct the gate. Notwithstanding the insistence of our attorney in the City of Mexico and various telegraphic communications insisting upon this approval being hurried, we were unable to obtain it until 12 months afterward, namely, the month of December, 1905.

"*Unprecedented River Conditions.*—In the meantime serious trouble had begun. We have since been accused of gross negligence and crimi-

nal carelessness in making this cut, but I doubt as to whether any one should be accused of negligence or carelessness in failing to foresee that which had never happened before. We had before us, at the time, the history of the river as shown by the daily rod readings kept at Yuma for a period of twenty-seven years. In the twenty-seven years there had been but three winter floods. In no year of the twenty-seven had there been two winter floods. It was not probable, then, in the winter of 1905, that there would be any winter flood to enlarge the cut made by us, and without doubt, as it seemed to us, we would be able to close the cut before the approach of the summer flood by the same means that we had used in closing the cut for three successive years around the Chaffey gate at the head of the canal.

"During this year of 1905, however, we had more than one winter flood. The first heavy flood came, I believe, about the first of February, but did not enlarge the lower intake. On the contrary, it caused such a silt deposit in the lower intake that I found it necessary, after the flood had passed, to put the dredge through in order to deepen the channel sufficiently to allow enough water to come into the valley for the use of the people.

"This was followed shortly by another heavy flood that did not erode the banks of the intake, but, on the contrary, the same as the first, caused a deposit of silt and a necessary dredging. We were not alarmed by these floods, as it was still very early in the season. No damage had been done by them, and we still believed that there would be no difficulty whatever in closing the intake before the approach of the summer flood, which was the only one we feared. However, the first two floods were followed by a third, coming some time in March, and this was sufficient notice to us that we were up against a very unusual season, something unknown in the history of the river as far back as we were able to reach; and, as it was now approaching the season of the year when we might reasonably expect the river surface to remain at an elevation that would allow sufficient water for the uses of the valley to be gotten through the upper intake, we decided to close the lower.

"*Five Floods in One Season.*—Work was immediately begun upon a dam similar to the ones heretofore successfully used in closing the cut around the Chaffey gate. The dam was very nearly completed, when a fourth flood coming down the river swept it out. Work was immediately begun on another dam which was swept away by the fifth flood coming down during this winter season."

These closings of the by-passes or cuts around the Chaffey Gate were effected by throwing a barrier of brush across the cut and dragging earth over it with Fresno scrapers, pushing it into the water on the up-stream side, thus gradually rendering the barrier impermeable and then building it up as an earthen dam. In attempting to make

the closure here mentioned, in March a small pile-driver was rigged up on the end of the *Alpha* and one line of 8 by 8-in. pine timbers, 3 ft. apart, was driven across the opening about 3 000 ft. west of the river bank, and an 8 by 8-in. waling was bolted to each pile above the water surface. Brush fascines were then made up, and all the sand bags available—about 10 000—were filled in readiness. Simultaneously from each side, brush fascines with the brush ends up stream, were piled above the piling and weighted down with sand bags, making alternate layers of fascines and bags, until the water was confined to a 30-ft. channel in the center. This barrier was about 20 ft. thick up and down stream. The opening was then spanned with long cottonwood timbers and a similar brush-sand-bag construction was built upon them. The supporting timbers were then shattered with dynamite, letting the mass drop into the opening. At the same time a large quantity of brush was thrown in above and allowed to float into the opening to help close it. In this way the barrier across the opening was built above water and teams passed over it dragging in dirt from both sides, the flow being reduced so greatly that the dredge below it nearly went aground. With a few thousand more sacks of earth to place along the upper toe of the barrier, the work would have been successful. As it was, the structure was undermined, settled down, and eventually failed entirely.

In this attempt 10 000 sacks were used, 8 days' time with the dredge at $100 per day, and 225 men-days time of Indian labor at $1.50 per day. This makes the total cost of closing about $1 800.

Instructions were then given to move the dredge up close to the river bank, where the soil was thought to be better, and make another attempt. The current through the break, however, was too swift, and instructions were given to go up the old Main Canal to the upper Mexican intake to stop it, which was done, using the method which had failed below.

A similar method was used to throw the water through the Alamo Waste-gate on its completion in June, 1905, 3 months later, 30 000 sacks of earth being filled in readiness and every one used. This barrier dam was thrown across the channel carrying 2 500 sec-ft. of water and with a total or final head of 10 ft. This has always seemed to the writer to have been a most remarkable achievement, the only equipment at hand being a skid pile-driver and Fresno scraper teams.

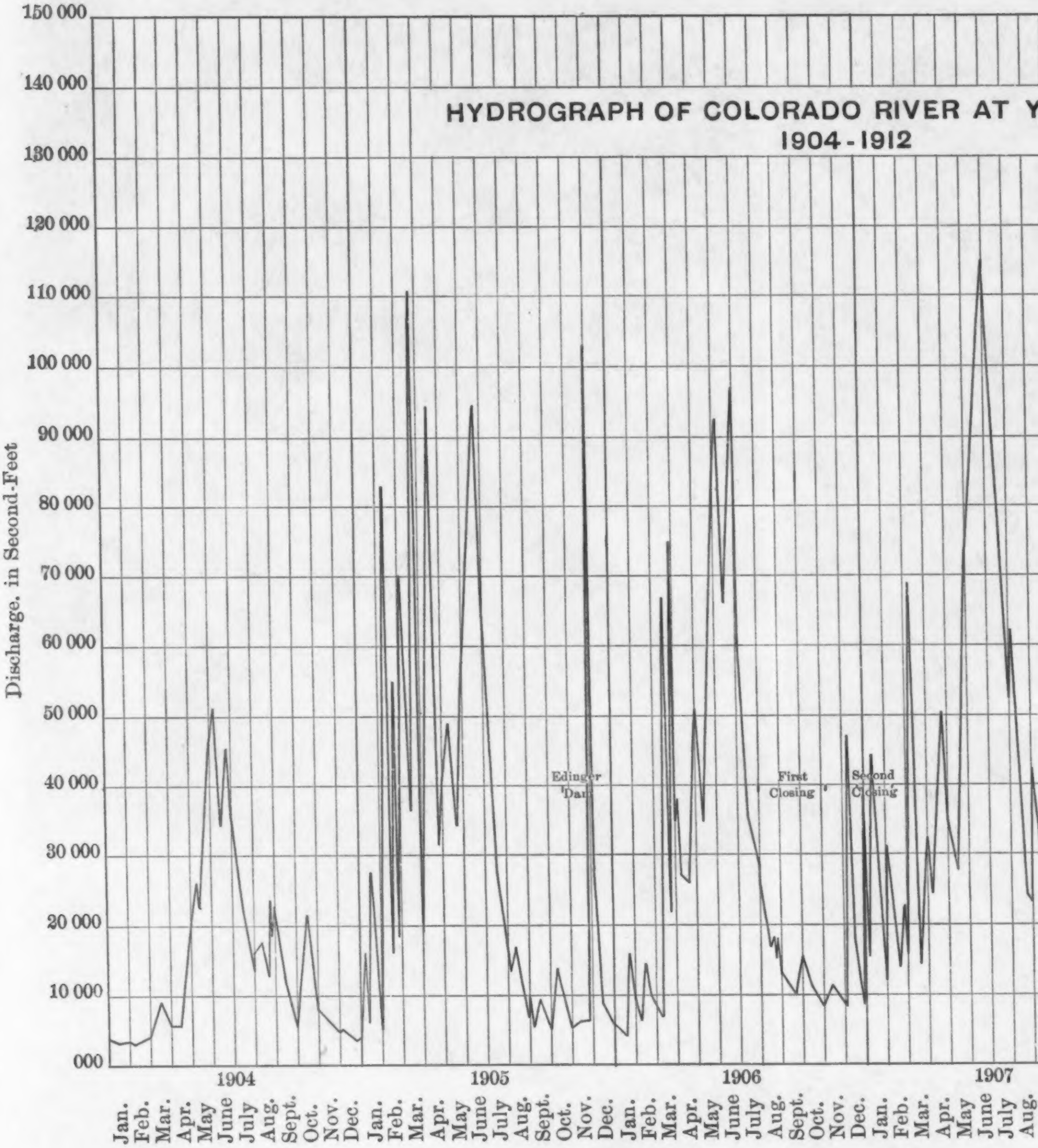
To resume Mr. Rockwood's narrative:

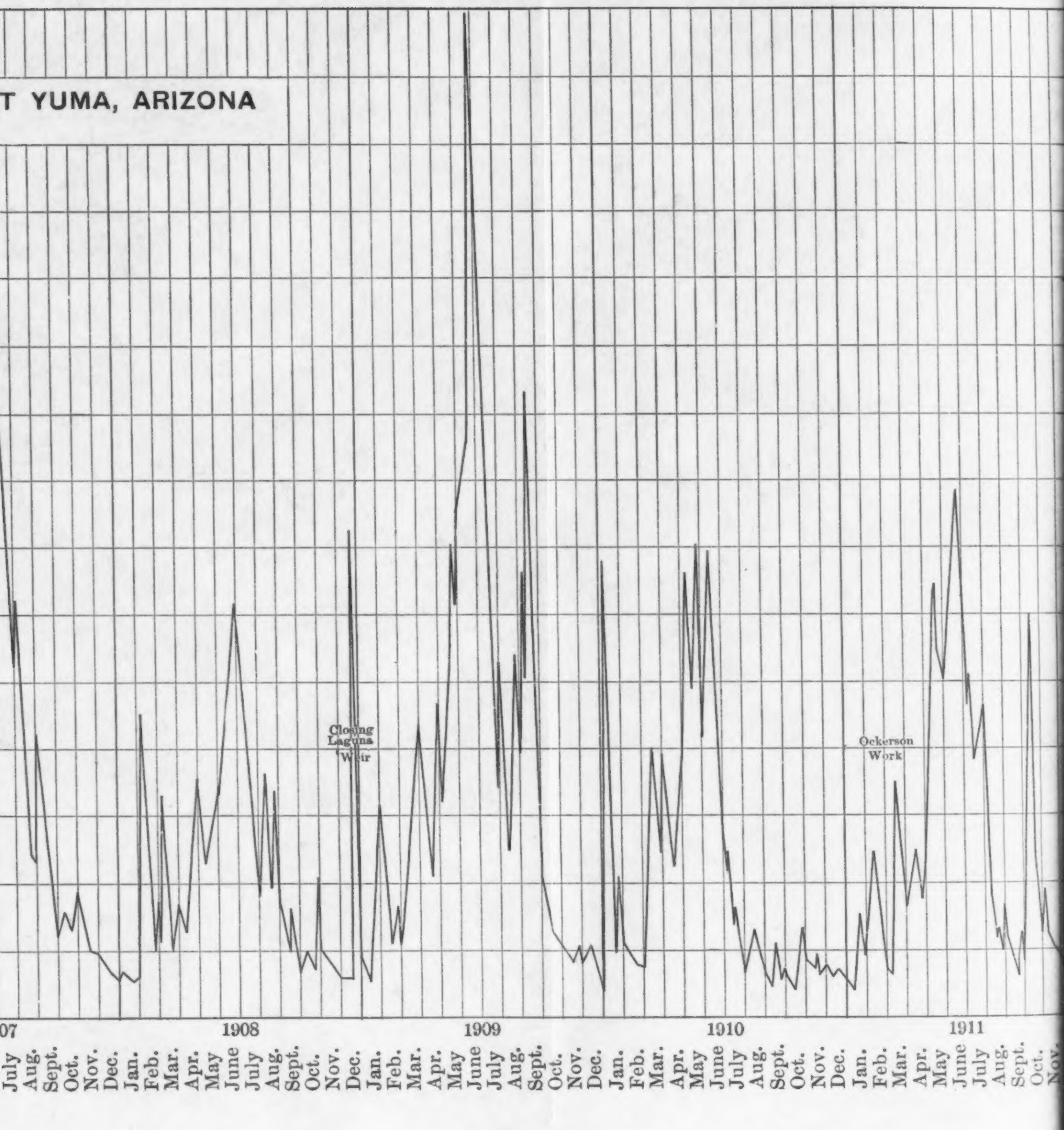
"About this time, I left for the East and at the earnest solicitation of the Imperial Water Company No. 1, which agreed to advance $5 000 for the effort, a third attempt to close the break was made under the direction of Mr. C. N. Perry and the superintendent of Imperial Water Company No. 1, Mr. Thomas Beach. On my return from the East, on the 17th of June, I found them heroically attempting to stop the break, with the waters so high in the Colorado that all of the banks and surrounding lands were flooded, and I immediately stopped the work as we realized fully that nothing could be done until after the summer flood had passed.

"*The Colorado on a Rampage.*—At this time, the lower intake had been enlarged from a width of about 60 ft., as originally cut with the dredger, to a width of possibly 150 ft., and it did not then seem probable that the Colorado River would turn its entire flow through the cut, but as the waters of the river began to fall, the banks of the intake began to cave and run into the canal, the banks of the canal below the intake fell in and, as known by most of the residents of the valley, the entire river began running through the canal and into the Salton Sea in the month of August of this year of 1905."

Plate CIX shows the discharge at Yuma to have been an unprecedented sequence of floods from the Gila water-shed. Indeed, the precipitation throughout all that region traversed by the Southern Pacific line from Yuma to very near El Paso during this period was quite without precedent. Track ballasted with local material, which had always proved satisfactory, was during this year the despair of the entire Maintenance of Way Department, and for months trains were allowed to go over it only at half speed and with lurchings of the coaches and Pullman sleepers like ships at sea.

Mr. Rockwood's statement gives a very fair presentation of the matter as he viewed it. The writer is perhaps as well aware as any one that the river was diverted through this cut into the Salton Sea, and when he first inspected the situation in August, 1905, he felt, like practically all other engineers who gave the matter cursory consideration, that making this cut was a blunder so serious as to be "practically criminal." After 4 years of more or less bitter experience with the region, he is perfectly convinced that, matters having gotten into such condition, making the cut was absolutely imperative and by all means should have been done. The difficulty had not been any tendency whatever to divert the entire river, but—very much to the contrary—


HYDROGRAPH OF COLORADO RIVER AT Y
1904-1912
Discharge, in Second-Feet
150 000
140 000
130 000
120 000
110 000
100 000
90 000
80 000
70 000
60 000
50 000
40 000
30 000
20 000
10 000
000
Edinger Dam
First Closing
Second Closing
1904
1905
1906
1907
Jan.
Feb.
Mar.
Apr.
May
June
July
Aug.
Sept.
Oct.
Nov.
Dec.


T YUMA, ARIZONA
Closing Laguna Weir
Ockerson Work
07
1908
1909
1910
1911
July Aug. Sept. Oct. Nov. Dec. Jan. Feb. Mar. Apr. May June July Aug. Sept. Oct. Nov. Dec. Jan. Feb. Mar. Apr. May June July Aug. Sept. Oct. Nov. Dec. Jan. Feb. Mar. Apr. May June July Aug. Sept. Oct. Nov. Dec. Jan. Feb. Mar. Apr. May June July Aug. Sept. Oct. Nov.

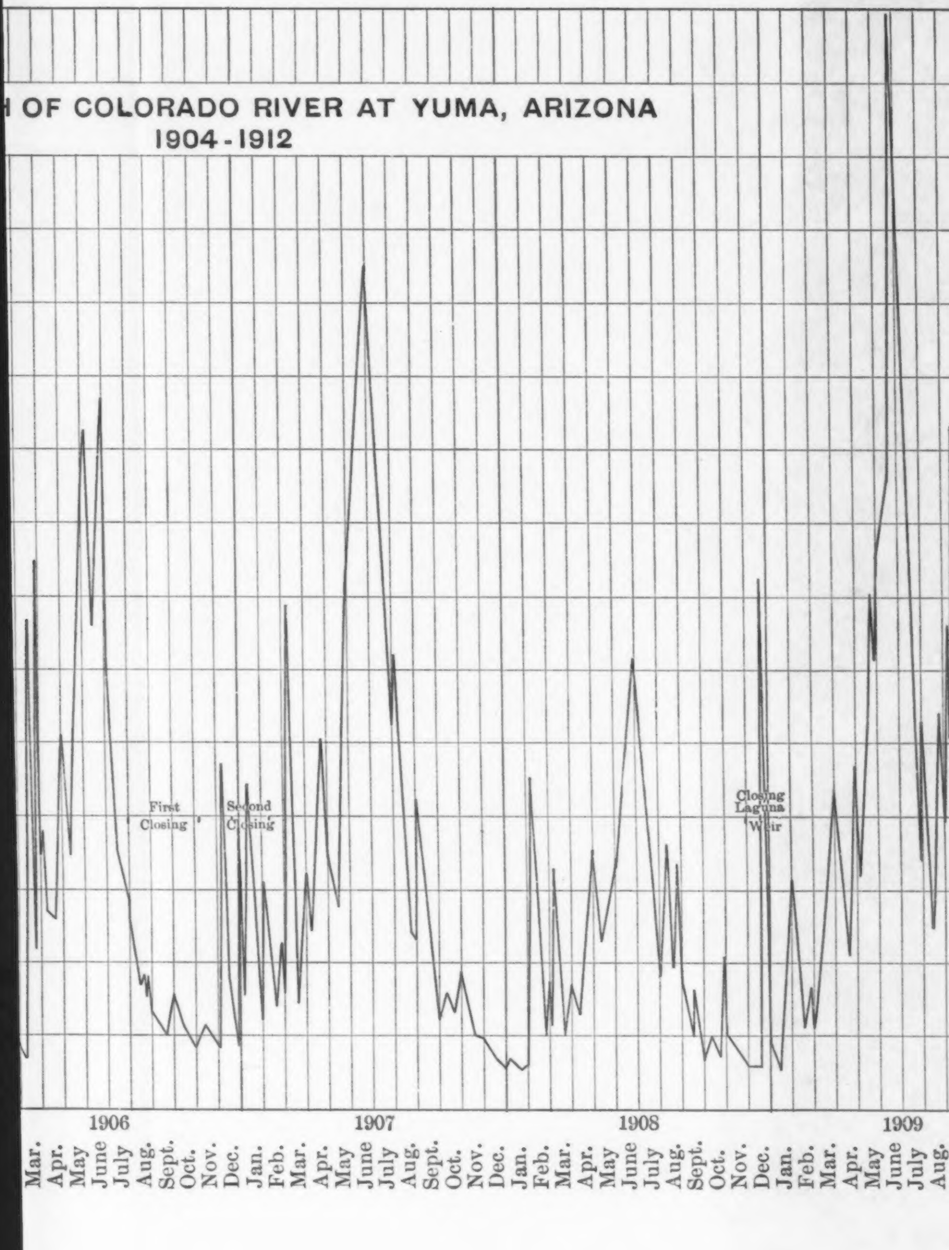
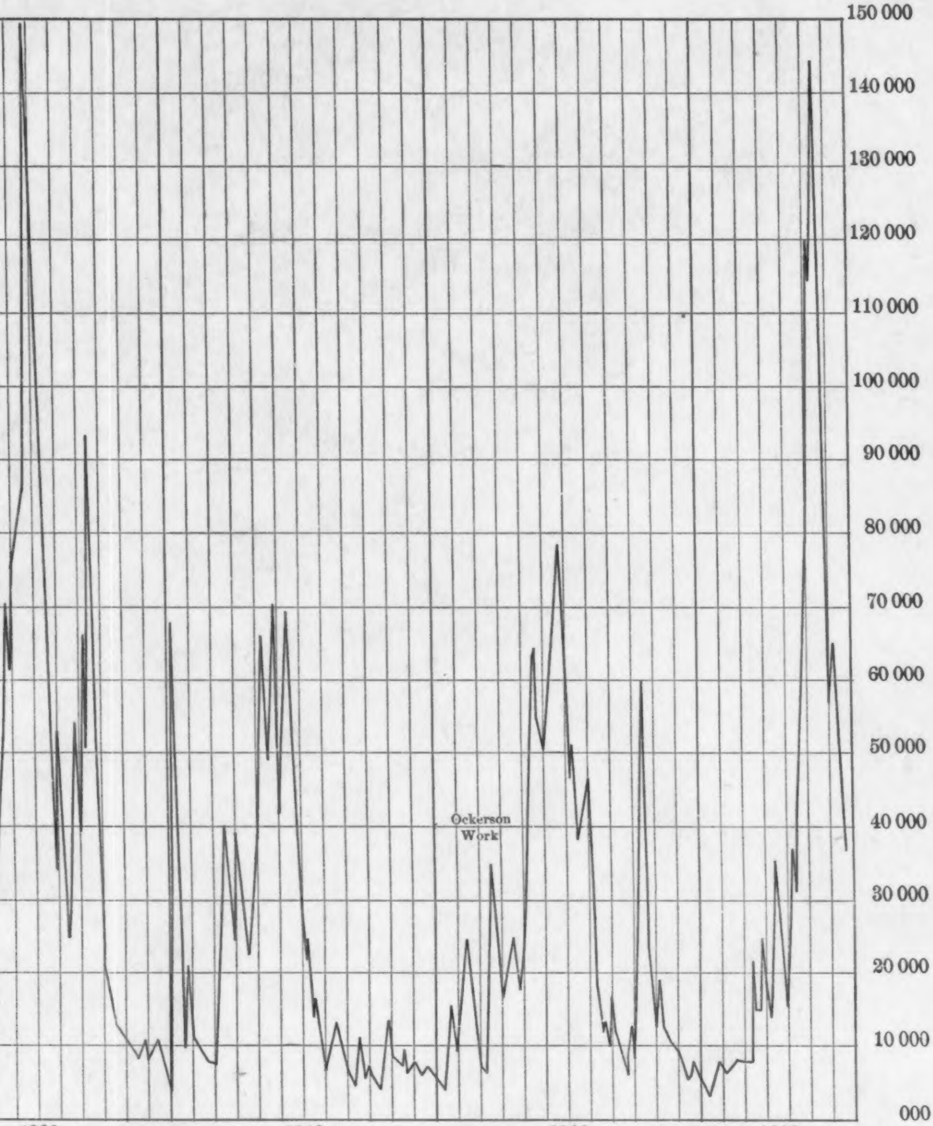

H OF COLORADO RIVER AT YUMA, ARIZONA
1904-1912
First Closing
Second Closing
Closing Laguna Weir
1906
1907
1908
1909
Mar. Apr. May June July Aug. Sept. Oct. Nov. Dec. Jan. Feb. Mar. Apr. May June July Aug. Sept. Oct. Nov. Dec. Jan. Feb. Mar. Apr. May June July Aug. Sept. Oct. Nov. Dec. Jan. Feb. Mar. Apr. May June July Aug.

PLATE CIX.
PAPERS, AM. SOC. C. E.
NOVEMBER, 1912.
CORY ON
IRRIGATION AND RIVER CONTROL,
COLORADO RIVER DELTA.



to induce enough water to go that way. Up to that time, a head-gate to prevent too great a quantity of water from entering the canals was of far less importance than some means of maintaining their carrying capacity. That a head-gate should have been provided is, of course, self-evident. It would have been utter folly, however, to have put a flash-board gate of any type directly in the diverting channel, because of the drift which would have accumulated against it. Nothing less than a structure containing immense openings could have been used without insuring that the cut would be choked up. This type of construction was practically unused in western irrigation works at the time, and would have cost a great deal of money, therefore, considering the financial condition of the C. D. Co., it is plain that the only practical thing would have been a gate, not in the cut itself, but in a by-pass, and built with the idea of closing the by-pass on the approach of the summer floods and using this gate as much as possible. It was not alone the straitened financial condition of the C. D. Co. and the situation generally in which it found itself which resulted in there being no permanent diversion works put in; two other important factors entered. The first was the practical change of management, from a construction point of view especially, to the Chaffeys in the summer of 1900; back to Mr. Rockwood in February, 1902, and internal difficulties in the C. D. Co. late in 1904. The second—indirectly connected with the first—was the hesitancy of the management to provide permanent head-works before the technical men in the corporation had agreed as to what the situation demanded. The real mistakes was not in "putting all the eggs in one basket," but in not "then watching that basket." Obviously, no one could be responsible for doing such a thing without realizing the need for watching it most carefully and being fully prepared to take most aggressive action should occasion arise.

Southern Pacific Loan.—Early in January, 1905, it occurred to the management of the C. D. Co. that the phenomenal development of traffic furnished to the Southern Pacific Railroad by the Imperial Valley warranted the hope of financial assistance from that corporation. Mr. Julius Kruttschnitt, Director of Maintenance and Operation of the Harriman Lines, declined to consider the proposition, but Mr. Harriman, on being approached, was at once interested and ordered an investigation and report. As a final result, the Southern Pacific

Company agreed to loan \$200 000 on condition that 6 300 shares of the capital stock be placed in the hands of a trustee to be named by the Southern Pacific Company until the loan should be repaid, and taking over the management of the property until that time. Accepting a loan under such conditions was seriously objected to by a large part of the company's stockholders, but at the annual meeting in Jersey City, in May, a board was elected pledged to the arrangement. On June 20th, 1905, the Southern Pacific Company took over the management of the property, with Mr. Epes Randolph, President of the Associated Harriman Lines in Arizona and Mexico, as President, and Mr. W. J. Doran, of Los Angeles, as the trustee mentioned in the contract. Both these gentlemen are still acting in these respective capacities. When the loan was arranged, and even when it was finally consummated, the railroad officials in San Francisco and the East did not consider the conditions along the river worthy of serious concern.

Mr. Rockwood was retained temporarily as Assistant General Manager and Chief Engineer, as members of the Southern Pacific management were entirely unfamiliar with the affairs of the C. D. Co.

Fourth Attempt to Close the Break.

As soon as the summer flood of 1905 began to recede, work was started. Immediately opposite the lower intake was an island, later dubbed Disaster Island, about $\frac{5}{8}$ mile long and $\frac{1}{4}$ mile wide, consisting really of a sand bar on which quite a growth of cottonwood and arrow weed had accumulated. A line of piling, 12 ft. from center to center, was driven from the upper end of this island to the Mexican shore, as shown in Fig. 12, and between this piling was woven barbed wire and brush. The theory behind this work was that, by spreading over a great width the water passing down the west channel and into the lower intake, a sand bar would be created, thus choking off the flow and gradually forcing all the water into the east channel. On July 15th about one-third of the river flow was going down the old channel and two-thirds toward the Salton Sea, and the result of this endeavor was still problematical. By August 1st a bar, approximately 2 800 ft. long, had been formed, but there was an opening, approximately 125 ft. long, through which the rush of water was too great to be controlled with the means at hand, and the work was abandoned. Up to this date, about \$30 000 had been expended on the four endeavors to close the break.

Various Suggestions for Handling the Situation.—At this time it was evident to all that the low-water flow of the Colorado would be entirely diverted into the old Alamo overflow channel and thence to the Salton Sea. The elevation of the water surface at the head of Disaster Island, with a flow in the river of 10 000 sec-ft., was approximately 100 ft., while the bottom of the Salton Sea is approximately — 287 ft., making the total fall in that direction 387 ft. The distance was about 95 miles by the watercourses, so that the average fall was 4.01 ft. per mile. Toward the Gulf the fall was 100 ft., and the distance to tide-water was approximately 80 miles, or a fall of 1.25 ft. per mile. The continually diminishing quantity of silty water going down the old channel as the summer flood receded was constantly raising the bed along that direction, the action being rapid enough to be noticeable almost daily. In all probability there were about 6 months ahead during which the flow of the water would be low, and before this period should elapse the river must be re-diverted or the consequences would be most serious.

The plant and the salt deposits of the New Liverpool Salt Company in the bed of the Salton Sea were already entirely submerged, the water covering about 100 000 acres, with a maximum depth of about 16 ft. Except for the increase of depth and the consequent increase in the length of time this property would be shut down, no additional damage was really being done at this point. Indeed, 14 years earlier, this property was covered to a depth of 6 ft. by the great flood of February, 1891, and the summer flood following, and in all probability a similar and greater inundation would have resulted from the excessive floods during the spring and summer of 1905 had the C. D. Co. never constructed any works along the river. The rising waters of the Salton Sea were threatening the tracks of the Southern Pacific Railroad along the east side of the sink, and the officials of the Los Angeles Division were clamoring for aggressive action. The higher officials of the company, however, were not yet very much perturbed. On the other hand, the Alamo channel was being enlarged and deepened, to the very great benefit of the C. D. Co., and the irrigation system of Imperial Valley, because the insufficient carrying capacity of this channel and the heavy silt deposits therein constituted a serious menace to the entire project.

To close the lower intake entirely meant obtaining all the water

required for the irrigation of Imperial Valley through the 4 miles of badly silted Main Canal lying between it and the upper intakes, and this was out of the question. Even with large sums of money, which might be obtainable from the Southern Pacific interests, machinery could not have been bought, assembled, and put into operation in time to have permitted the delivery of more than enough water to supply the inhabitants and live stock of the valley with drinking water if the river flow should be reduced to 6 000 or 7 000 sec-ft. Imperial Valley at that time consisted of at least 125 000 acres under cultivation, five towns with an aggregate population of 2 500 people, and a rural population of approximately three times that number. There were, perhaps, 100 000 head of hogs, 50 000 head of cattle, and other live stock in proportion.

Many plans were suggested, from this time, August 1st, 1905, until the break was finally closed in 1907. Many of these, of course, were thoroughly absurd, and came from cranks and people who had not the faintest conception of the conditions. Indeed, almost the only people who appeared to be able to see that the problem was not merely one of shutting off the lower intake were the engineers of the C. D. Co. and a few of the well-informed men in Imperial Valley. Representatives of the New Liverpool Salt Company, the Southern Pacific Company, various departments of the United States and Mexican Governments, and the general public, all joined in demanding aggressive action to stop the menace of a new Salton Sea.

Such suggestions were addressed to Mr. Harriman and to nearly every other official of the Southern Pacific interests, and to Mr. Randolph and other authorities of the C. D. Co. Ultimately, most of these found their way to the writer; they constitute a most interesting collection. It is not profitable to mention more than four of these suggestions, which may be designated the Laguna Weir Plan, the Concrete Head-gate Plan, the Rockwood Head-gate Plan, and the Barrier Dam Plan. Edwin Duryea, Jr., M. Am. Soc. C. E., also offered to close the break according to a plan, which, however, he declined to outline.

The Laguna Weir Plan.—The Laguna Weir Plan consisted in abandoning operations for the time being at the scene of the break; concentrating all efforts on the completion, at the earliest possible date, of the Laguna Weir, which was being built by the U. S. Reclamation

Service; building a canal thence passing Pilot Knob and intersecting the break from $\frac{1}{2}$ to $\frac{3}{4}$ mile west of the Colorado River, this canal to have a capacity equal to the low-water flow of the river; then diverting all the river water through this canal; finally, to build a dam across the intake between the canal junction and the river bank in still water. The Laguna Weir was actually completed in the early spring of 1909, just before the annual record flood of that year. It is not clear just how its completion could have been essentially hurried. Had this plan been followed, the Colorado would have emptied into the Salton Sea for 3 years longer than it actually did, and during this time 55 000 000 acre-ft. of water went by Yuma, only a very small portion of which would have gone down the old channel to the Gulf. This would have raised the water in the Salton Sea to the 180-ft. contour, with the effect of drowning out a large area of cultivated land in the Coachella Valley and forcing the abandonment of 60 miles of main line track by the Southern Pacific Railroad.

These effects, however, would have been of relatively minor importance. The irrigation system of Imperial Valley would have been strained far beyond the breaking point in several places, while the cutting back in New River would unquestionably have reached the Alamo channel and lowered the water therein far beyond the point where any could have been gotten into the Imperial Valley by gravity. This, of course, would have meant the depopulation of that region, an appalling result, without parallel in history.

The Laguna Weir Plan is thus seen to have been impracticable, and no one actually connected with the work gave it serious consideration. Nevertheless, it was urged on Mr. Harriman by Mr. C. D. Walcott, then Director of the United States Geological Survey and of the Reclamation Service, and Mr. Harriman considered it for quite a time.

Concrete Head-gate Plan.—The Concrete Head-gate Plan was put forward by the late James D. Schuyler, M. Am. Soc. C. E., who acted as Consulting Engineer of the C. D. Co. from July, 1905, to June, 1906. It consisted essentially of building a reinforced concrete and steel head-gate on the Pilot Knob site, where solid rock foundation could be secured, such gate to be able to carry the low-water flow of the river; and then, from this head-gate down to its junction with the crevasse, to enlarge the canal to a similar capacity. This, it was considered, would permit the diversion of all the water through the head-gate and

canal, leaving the river below, and consequently the break itself, dry. The underlying idea was somewhat similar to that of the Laguna Weir Plan, except that it contemplated only 4 miles of canal enlargement and a diversion structure which could be completed in 3 or 4 months, instead of 3 years.

This plan involved the construction of permanent head-gates on rock foundation at Pilot Knob, so long contemplated; and the construction and equipment of a dredge with which the requisite 4 miles of canal could be dug economically and quickly. The idea was adopted in a tentative way in September, 1905, approximately 90 days after the Southern Pacific Company undertook the management of the C. D. Co., and Mr. Schuyler was instructed to proceed with the preparation of plans for the head-gate, while Mr. F. S. Edinger, under whose direction the Edinger Dam was built, arranged for the dredge. At the suggestion of the Golden State and Miners Iron Works, of San Francisco, the clam-shell type, with 150-ft. boom and 5-cu. yd. bucket, was selected. Work was begun on the concrete head-gate on December 15th, 1905, and contracts for the clam-shell dredge were arranged a few weeks later.

One of the chief recommendations of this plan was that the constructions, in large measure, would be permanent. It was assumed that, while perhaps the maximum quantity of water which would have to be diverted for the irrigation of Imperial Valley would never exceed 5 000 sec-ft., a gate twice as large would not have any particular disadvantages in its maintenance or operation. It was urged, further, that this arrangement of diverting structure and large canal would be available in case of future breaks, should any ever occur.

The difficulty about the plan was that, regardless of the size of the gate, enlarging the 4 miles of canal to carry 10 000 sec-ft. within sufficient time to afford reasonable relief was a very serious problem, while the capacity of this canal would be reduced so quickly by silt deposition that its use in case of future breaks would be out of the question. Furthermore, to insure the diversion of all the water in the river, required a canal cut considerably below the water-table in the ground through which it would have to pass, and large patches of quicksand occur so frequently in this region that it would be folly to hope to miss all of them. Such patches would cause the inflow of material from the sides and the bottom to a serious extent.

Mr. Rockwood's Plan.—Mr. Rockwood urged the necessity of a rapid re-diversion, not so much because of the effect on the Southern Pacific tracks along the Salton Basin as because he understood the critical condition at a number of points in the irrigation system of Imperial Valley, and that the severe strain could not be withstood successfully for very many months. His suggestion, made in August, 1905, was to put in, immediately beside the break, a wooden **A**-frame, flashboard head-gate, capable of passing the low-water flow of the river; with dredges to dig channels from the break to the gate both above and below; divert the water through this by-pass and gate with a piling-brush-sandbag barrier dam; complete the dam as an earth fill across the break, and build levees both up and down stream as far as might be necessary; then close the gate to such an extent as would admit only enough water to supply the irrigation needs. This plan was approved, and work was started on September 20th. It was abandoned completely 3 weeks later; was again approved on December 15th, 1905; and was carried out until the gate construction failed, in October, 1906. It was daring only in its size and the foundation of so important a structure on alluvial soil, and it would have resulted in permanent diversion works on Mexican soil—where, by all means, they should have been, originally, and as contemplated in the Mexican concession, granted in 1904.

The Barrier Dam Plan.—The Barrier Dam Plan consisted in throwing a barrier dam of some sort across the crevasse and raising the water surface above it sufficiently high to throw all the discharge of the river down the old channel to the Gulf. The usual type of dam was suggested, of piling and brush mattresses of fascines weighted down by sandbags. This method seemed to its proponents to afford opportunity for decreasing the quantity of water diverted in the minimum time, and neglected that side of the problem which required the furnishing of water for the Imperial Valley. The best plan for a structure of this type was that put forward by Mr. Edinger, and worked on under his direction from early in October until its destruction by the great flood of November 29th, 1905.

Fifth Attempt to Close the Crevasse.

Mr. Rockwood presented his plan to Mr. Randolph and Mr. Schuyler, and they, as well as several engineers of the Southern Pacific Company, approved of its trial. Plans were hurriedly worked out for a wooden

A-frame, flash-board gate, 120 ft. long, with a concrete floor, and founded on piles. Rush orders for materials were placed, and the first shipments left Los Angeles on August 7th. It was fully expected to have the structure completed by November 15th.

The original intention was to construct the gate in a by-pass to be excavated by the dredge *Alpha* on the south side of the intake, but examination showed an unfavorable foundation, as the ground slid into the cut so rapidly that the dredge was almost caught and held by it. The plans, therefore, were changed, and it was decided to construct a by-pass on the other side of the break; force all the water through this by-pass; and then build the structure where the intake had been, thus saving both time and money in the excavation. The break at this point was about 300 ft. wide—just about the length of excavation required for rapid and successful construction. The dredge was put to work on this by-pass, and no difficulty whatever was found in making the 700-ft. cut required. The plan worked very well, and a large part of the water began to go that way at once. Work was begun on the up-stream side of the coffer-dam, the idea being that, when all the water was diverted through the by-pass, another earthen dike would be thrown in, about 250 ft. below the first, and thus make the coffer-dam for the gate construction. In this way, the second dam would be built in still water and in very short order with the dredge.

At this time—about September 15th—it became evident to Mr. Rockwood that he could not attend to the business affairs of the company properly and remain in personal charge of the work along the river. It seemed easier to find some one capable of completing the gate in accordance with the plans outlined than to find any one qualified to handle the corporation's affairs. Mr. Edinger was selected, as he, until June, 1905, had been for many years Superintendent of Bridges of the Southern Pacific System, and had had very large experience in coffer-dam work. About 3 months previously he had left the Southern Pacific Company and entered the contracting firm of Shattuck and Desmond, of Los Angeles and San Francisco. About September 20th, Mr. Edinger and Mr. Rockwood went over the ground and the plans together, and Mr. Edinger commenced the work.

The records show that, about October 1st, the river usually rises 2 or 3 ft., principally due to rains on the water-shed of the Gila River and Bill Williams Fork. This year was no exception, and the slight rise

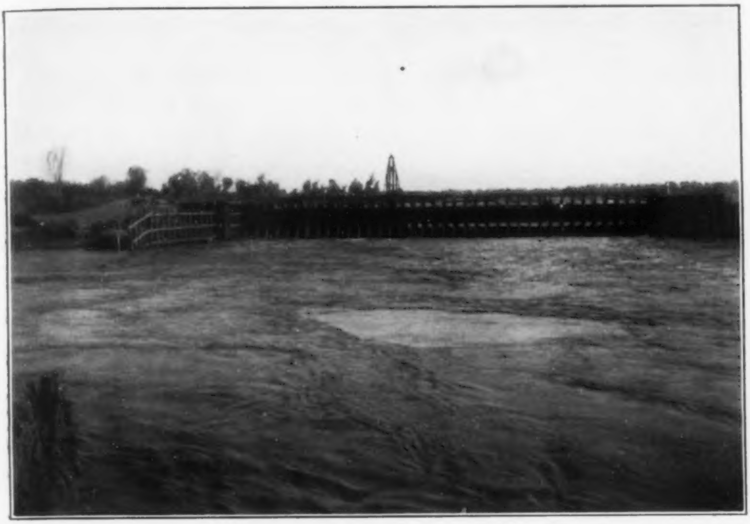


FIG. 1.—ROCKWOOD HEAD-GATE, OCTOBER 6TH, 1906, PASSING 12 000 SECOND-FEET.

FIG. 2.—EDINGER DAM, NOVEMBER 8TH, 1905. BRUSH AND WIRE MAT IN FOREGROUND, READY TO BE PLACED.

about October 1st shook Mr. Edinger's confidence in the plan. He quickly outlined a barrier dam plan to Mr. Randolph, who approved of it, and work was shifted to it at once. This plan consisted of constructing a piling and brush dam across the west channel between the head of Disaster Island and the Mexican shore, a distance of about 600 ft., and it was expected that the river would all be turned down the east channel before a gate could even be put in. All material was at once removed from the gate site, and work was rushed on the construction of what is locally known as the Edinger Dam.

This plan of handling the situation, in addition to shutting off all water flowing into the Imperial Valley through the lower intake, was seriously defective in that even a short flood sufficiently great to send any water overbank in the immediate vicinity of the dam—and that would require much less water than usual on account of the silted-up condition of the whole river bed below the break—would in a few hours result in cutting the channel around the end of the structure and entirely shunting it. Indeed, such a re-diversion was exactly what took place a little more than a year later, when the waters broke under the levee, ½ mile south of the Hind Dam, in December, 1906. Had the Edinger Dam been entirely successful and completed on November 15th, such re-diversion would have been caused by the terrific flood of November 28th, and so on; the hydrograph, Plate CIX, shows a number of floods sufficiently great to have done this. Indeed, at this time, no one seems to have realized that a large, deep, and efficient channel had been created from the Lower Heading westward for many miles, and that future safety demanded, not only closing the intake, but an elaborate system of levees reaching miles both up and down stream.

The plan of the Edinger Dam consisted in driving rows of piling and filling the interstices with brush mattresses and fascines. The idea behind it was essentially similar to that of the work abandoned about August 1st. To have been successful, the construction would have had to withstand a head of from 8 to 10 ft. However, on November 29th, when a head of 35 in. had been obtained, a terrific flood came down from the Gila, reaching a gauge height of 31.3 ft. at Yuma and a discharge of 115 000 sec-ft. Large quantities of drift were carried by the flood-waters. This drift collected against the Edinger Dam in great quantities, and a large volume of water went down the east side of the island and the old channel. Before the flood had reached its peak, the dam

started to give way, and in an incredibly short time was practically destroyed. When the river had again fallen, the old channel was silted up higher than before, the new channel was scoured still deeper, and when the flow of the river had decreased to 17 500 sec-ft. all the water was again going toward the Salton Sea.

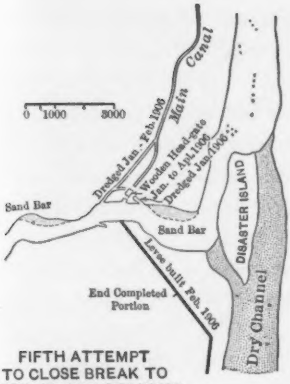
The flood not only wrecked the dam, but carried away practically all the material on the ground, and, after it receded, side-cutting along the west side of Disaster Island began to take it away rapidly. It was soon obvious that it would be folly to resume work at that location, and it was decided that the piling-brush-sandbag barrier dam method was not to be given further consideration.

So much water went through the break to the valley, at the failure of this dam, that the *Alpha* was sent to the Quail River and put to work cutting a channel southward in the hope of diverting a large part of the flow into the Paredones and thence *via* Volcano Lake into the Gulf. It was an endeavor to divert a large part of the water from an old overflow channel on the north side of the delta cone into an overflow channel on the south side thereof. It had little result, however, and the Quail River cut soon closed itself.

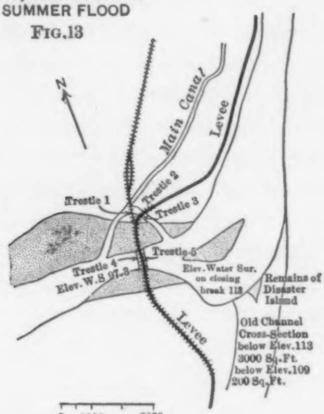
On October 15th there were 20 white men and 25 Indians at work on the Edinger Dam; on November 1st, 42 white men and 50 Indians; on November 10th, 106 white men and 65 Indians, and on November 29th, 250 white men and 80 Indians. Two steamboats with barges attached, and the relatively large barge *Silas J. Lewis,* with their crews, were also at work.

On the books of the company, the cost of the Edinger Dam is not thoroughly segregated from the expense incurred in the head-gate work up to the time of its abandonment for the barrier dam plan. The expenditures on it, however, were about $60 000, and the grand total to December 1st, 1905, was about $100 000.

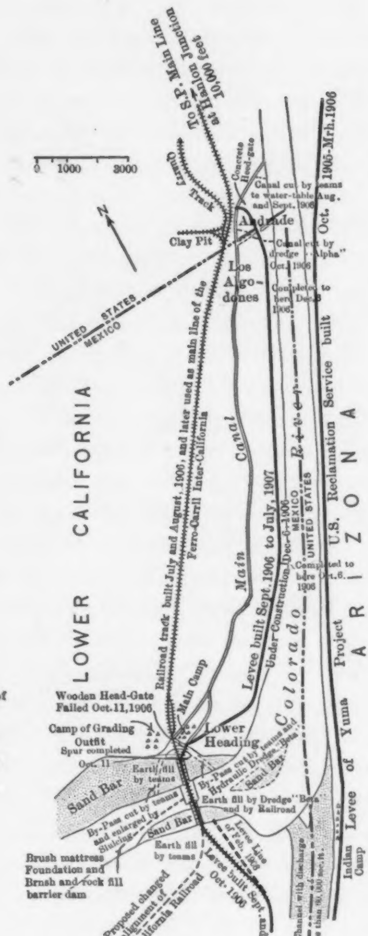
Concrete Head-gate.—The location of this interesting structure is shown on Fig. 14, where the granite point of Pilot Knob is near the right bank of the river. The general design was outlined by Mr. Schuyler, and the principles used, dimensions, elevations of flow, etc., were submitted to Messrs. Rockwood and Randolph, and approved by them. George S. Binckley, M. Am. Soc. C. E., then worked out the details and prepared the working drawings. Contracts for the structural steel and ironwork were let to the Llewellyn Iron Works, of Los



FIFTH ATTEMPT
TO CLOSE BREAK TO
APRIL 15, 1906-BEGINING
OF SUMMER FLOOD
FIG.13



SIXTH AND SUCCESSFUL
ATTEMPT TO CLOSE THE BREAK
OCT. 12 TO NOV. 4, 1906
FIG.15



FIFTH ATTEMPT TO CLOSE THE BREAK-
JULY 1, TO OCT. 11, 1906
AFTER SUMMER FLOOD
FIG.14

Angeles, and for the construction work to Mr. Carl Leonardt, also of Los Angeles, on November 25th, 1905. Time was made the essence of the contract, and Mr. Leonardt hurried the necessary equipment to the ground and began actual work 2 weeks later. Although it was expected to complete the gate ready for use within 90 days, the entire job was not finished until June 28th, 1906.

Type and Size.—The intake gate is doubtless the largest and most expensive irrigation canal head-gate in America. The design is a modification of the Taintor or radial-gate type, which has been used for many years for irrigation constructions in the Western States. This style of structure was adopted in order to obtain openings of maximum area easily and quickly opened or closed by one man. It had probably not been used in Calfornia, although a large wooden radial-gate had been built some years before at the head of the so-called Peoria Canal from the Gila River, near Gila Bend, Ariz. It was about 25 ft. high and 30 ft. wide, which is nearing the extreme for construction of that class. This wooden gate, however, was never used, as the dam across the Gila River was destroyed by flood soon after its completion. The maximum height of radial-gates and canal head-works in Idaho at the time was about 11 ft., and the water was not expected to rise to the top of the gates, the river level being controlled by other means.

Here, however, the extreme flood level is 19 ft. higher than the low-water level, so that gates at high flood time are subjected to great pressure. Sufficiently large vertical lifting gates would have required very heavy and massive piers, and the gate would have been very large and disproportionately high as compared with the width. These considerations caused the adoption of culvert openings between the piers for supporting a cellular structure of reinforced concrete, and thus admitting of loading the construction with gravel filling in the cells in order to get the required stability and weight at minimum cost. The gates were thus required to close culvert openings of minimum size, being in fact no larger than with the head at a uniform low-water height, although, of course, much heavier and stronger on account of the increased pressure at flood stages. There are eleven such culverts, each 10 ft. high and 12 ft. wide. In addition, there is a "navigation pass," the purpose of which was to permit passing a small gasoline launch through the gate. This navigation pass is practically useless because the mill race through it, when the difference in water level

above and below the gate exceeds 1 ft., precludes the idea of dragging a boat through it; indeed, no attempt has ever been made to use it. The floor of the gate is 98 ft. above sea level, according to the C. D. Co. datum, and 100.9 ft. according to the U. S. Reclamation Service datum. At the time, and until after the summer flood of 1909, the average low-water surface in the river was about 108 ft. The elevation of the flow line at the gate, therefore, was fixed so that the culverts would run full at low-water stage. The present low-water surface is about 105 ft.

The area of the eleven culverts in 1 320 sq. ft., and, with the water 1 ft. higher on the up-stream than on the down-stream side of the gate, their combined discharge would be 8 500 sec-ft. In addition, a large quantity of water would go through the navigation pass, which is 10 ft. 3 in. wide. When the water is 10 ft. above the top of the culverts, it is necessary to close the gates within 3.8 ft. of the bottom to hold the discharge through them down to 10 000 sec-ft., when the carrying capacity of the canal below is great enough to allow the water to get away.

The gate was designed to pass the entire low-water flow of the river—which it was assumed would certainly not exceed 10 000 sec-ft.—without any diverting dam in the river opposite it.

Cost of Structure.—Table 12 gives the cost of this structure, with the contract prices for excavation, concrete work, etc.

The cost of the gate, however, was considerably more, because Contractor Leonardt presented a claim insisting that the prices for earth and rock excavation named in his contract were agreed to by him on certain assurances made by Mr. Rockwood as to the character of the excavation which proved more difficult than expected. This claim was made as soon as Mr. Leonardt's representatives reached the ground, and Mr. Randolph permitted a change to a force account basis because of his desire to hurry the construction in every possible way. The earth excavation in this way cost 64 cents per cu. yd. and the rock \$2.06, thus increasing the figures by \$10 813, making a grand total of \$55 221.08.

Careful accounts were kept, and it was ascertained that the contractor made a profit of \$2 700 on the concrete, and \$741.50 on erecting the gates. What the earth and rock excavation should have cost is a matter of some, though slight, interest to the Profession, as these

would necessarily vary according to local conditions. As a matter of fact, with a good pumping plant, a mining nozzle or giant, a hydraulic elevator, and some pipe, the earth excavation could probably have been handled for 20 cents per cu. yd., and possibly less. Much of the rock was fairly soft, and could have been worked easily and cheaply, so that, had the contractor put in power drills and one or two long-boom derricks to handle the rock out of the cut, it is probable that the cost of such excavation would not have exceeded the contract price. The quantity of water entering the coffer-dam, or rather excavation pit, was surprisingly small.

TABLE 12.

GATE STRUCTURE.	
Earth excavation, 12 637.1 cu. yd. at $0.25	$3 159.28
Rock excavation, 5 700.81 cu. yd. at $1.00	5 700.81
Cement, furnished by company, 1 835 bbl. (Olsen, Gillingham, and Independence brands)	4 432.25
Concrete, labor, forms, sand, gravel, and rock, 1 204.83 cu. yd. at $9.00	10 843.47
Reinforcing steel bars, 25 722 lb. at 4 cents	1 965.16
Expanded metal for gate facings, 791 lb. at 4 cents	31.64
Allowance for 3 days' delay to contractor	102.50
Extras	807.07
	$27 042.18
Charges against contractor	271.70
Total cost of gate structure	$26 770.48
IRON AND STEEL WORK FOR GATE.	
Llewellyn Iron Works' original contract for twelve radial gates and one slide-gate (in navigation pass) f. o. b. Los Angeles	$12 000.00
Freight to Yuma, on 212 184 lb. metal in aforesaid gates at $1.25 per ton	132.60
Regulating levers, shaft, and gear (subsequent contract)	580.00
Erection of gates (Leonardt's contract)	1 500.00
Total cost of iron and steel work	$14 812.60
Plans, engineering, and superintendence, 6.7%	2 825.00
Total cost of head-works	$44 408.08

Fig. 10 gives a plan and elevation of this gate, and Figs. 1 and 2, Plate CXI, are views of the structure.

Purpose.—At the time this gate was designed, the money available for construction, through the Southern Pacific's connection and the loan of $200 000, justified the immediate construction of permanent

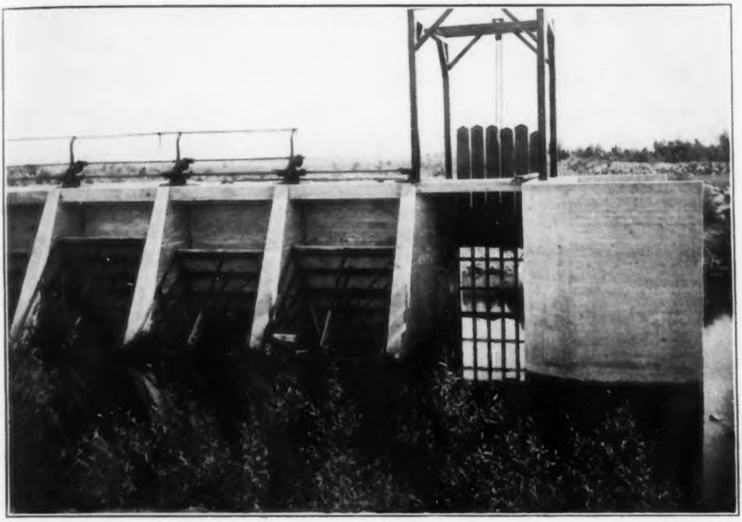


FIG. 1.—CONCRETE HEAD-GATE, JULY 10TH, 1906, SHOWING DETAILS OF GATES, NAVIGATION PASS, AND GATE AND ABUTMENT AT RIVER END.

FIG. 2.—GATE RAISING MECHANISM OF CONCRETE HEAD-GATE.

head-works, indeed, building this and the Alamo Waste-gate were the chief items for which the loan was made. Furthermore, the entire diversion of the river at the lower intake had shown the folly of trying to get along without them. The gate, while intended as a permanent diverting structure, was, nevertheless, primarily designed for use in endeavoring to re-divert the river; otherwise, of course, it would have been made much less than half as large. Actually, it played no part at any time in diverting the stream.

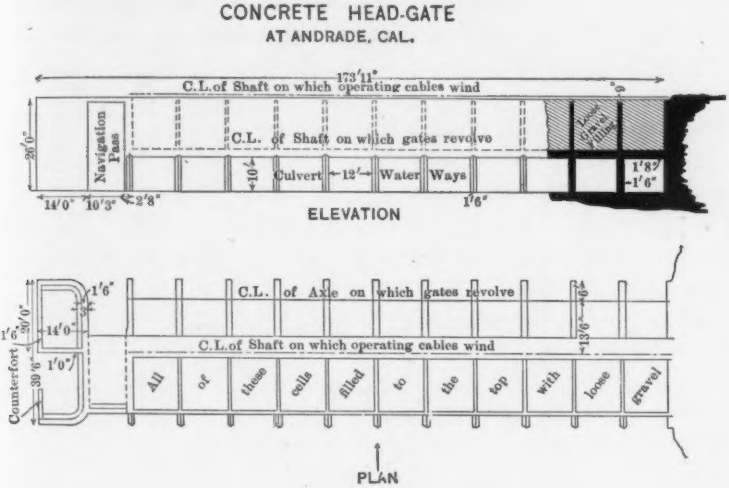


FIG. 16.

Operation.—The gate was actually put into operation on November 1st, 1906, when the water going through the break had been reduced to a quantity too small for the requirements of the valley. About 6 weeks later, the flood which caused the second break occurred, and resulted in an accumulation of drift on the up-stream side of the gate which choked up the underground culverts and practically put it out of commission. From that date to this the troubles caused by drift in the river, particularly at high-water periods, have been serious and often acute. Gates of this type, for head-works on a river carrying any drift to speak of, let alone as much as the Colorado often has, should be avoided. After considerable experience it is obvious that if permanent diversion of the water for the irrigation of the valley is not made on Mexican territory, then, whenever enough

money is available, it will be best to abandon the structure entirely and make diversion through gates similar to those in the sluice-ways of the Laguna Weir.

Aside from the type of gate for such a locality and stream, three unfortunate features in design became manifest. Chief of these was the fault that the drums on which the wire cables for raising the gates are wound are much too small. The gates themselves were designed for minimum weight with the necessary strength, and are not stiff enough, so that they tend to wedge unless exceedingly great care is taken. The net result is many broken cables. At one time only

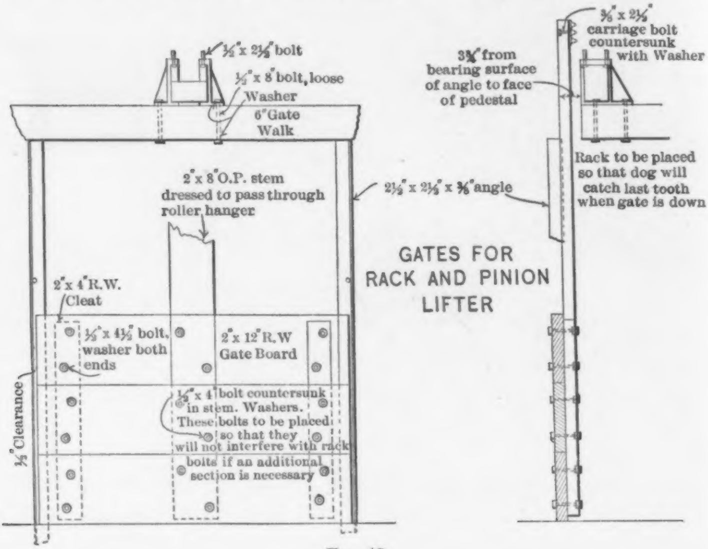


FIG. 17.

two of the eleven gates were in operation, some being clear down, some clear up, and some impossible to close entirely on account of driftwood under them. Fortunately, the *Delta* was near by and was used to raise the gates, so that new and strong plow-steel cables could be installed, replacing the original ones of Tobin bronze, ¾ in. in diameter, 19 wires to the strand. These plain steel cables corrode badly, of course, but still are much better than any galvanized iron ones of usable diameter.

Another bad feature of the design is the form of abutment built on the outer end of the gate. The writer has always been fearful

that water would find its way through the 10-ft. tongue of puddled earth which is the only barrier preventing water from getting around the end and shunting the gate entirely.

In September, 1906, a canal, from the river to the head-gate, was excavated by teams and Fresno scrapers. This intake was made 100 ft. wide at the bottom, with 2½ to 1 side slopes down as low as the underground water-table would permit. At about the same time the *Alpha* reached the Upper Heading and cut into the concrete gate excavation from the Main Canal below. The upper connection was wide enough, but the bottom was at least 6 ft. above the floor of the head-gate, and the down-stream connection was about 3 ft. above the floor of the gate and much narrower. These connections were widened and deepened to their present capacity by erosion, dredging, and blasting, as explained later.

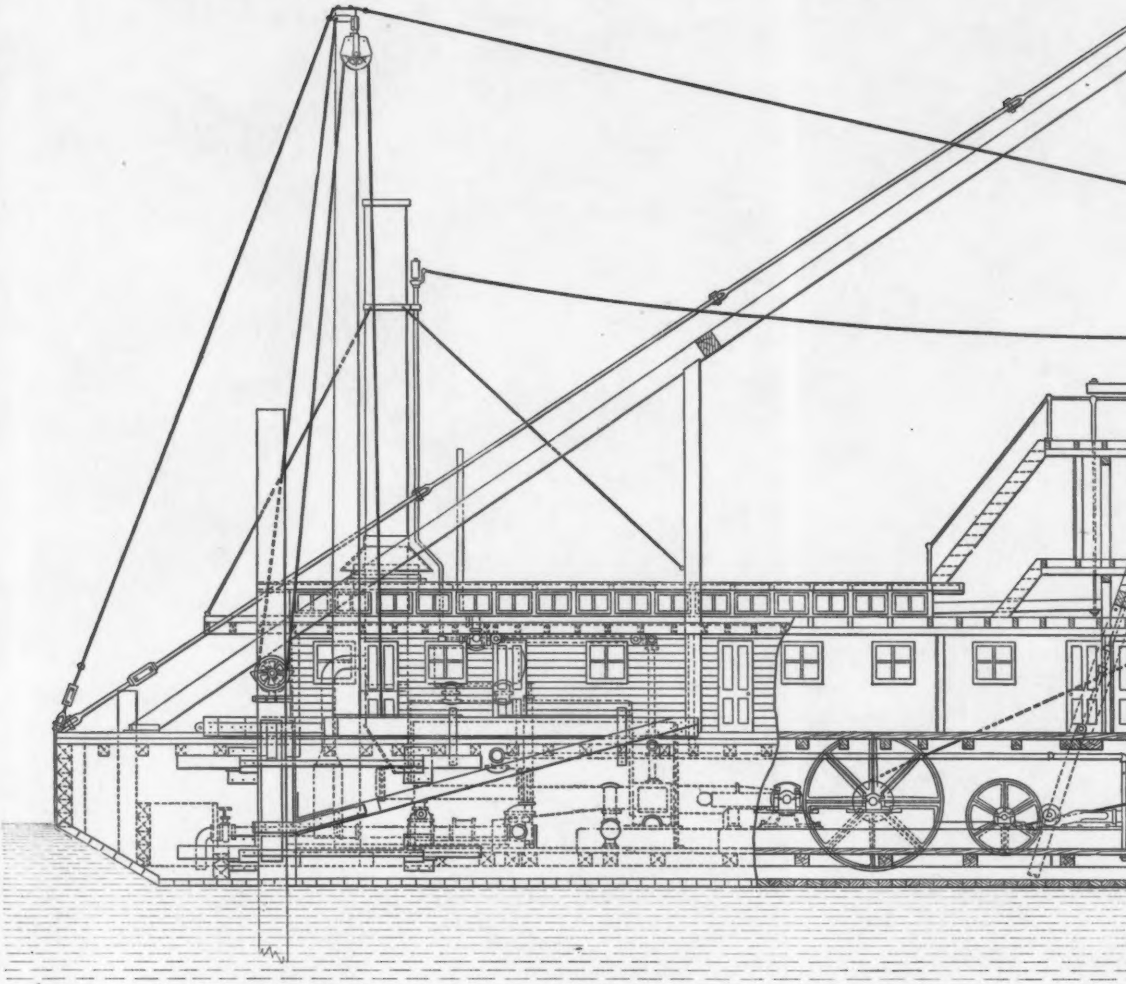
The Dredge, Delta.—The other element in the concrete head-gate plan of re-diversion was a canal from the head-gate to the break, a distance of approximately 4 miles. It was to be of sufficient size to carry the probable minimum flow of the river, 10 000 sec-ft. As it was obvious that this stretch of canal would have to be lower than the bed of the river all along the line, in order to permit of taking the entire low-water flow without a diversion dam in the river opposite the head-gate, a very large part of the cross-section to be excavated would be below the permanent water-table of the region. Therefore, some kind of excavating machinery which could handle large quantities of material under water had to be provided. It was taken for granted that the cheapest and quickest method of providing this waterway was to enlarge the existing Main Canal, although the writer thinks this was erroneous. The dipper dredge, *Alpha,* by almost continuous operation in this part of the course, had built up levees on both banks so high as practically to limit its future operation without flattening down these levees with teams and scrapers. Largely on the advice of Mr. Edinger, it was decided to construct a clam-shell dredge of the type used almost exclusively for levee building along the Sacramento River. Accordingly, a contract for machinery, and for plans, bills of materials, etc., of the hull, was entered into with the Golden State and Miners Iron Works, of San Francisco, which makes a specialty of clam-shell dredge machinery, construction, and even operation, on the Pacific Coast. This contract was closed on January 10th, 1906. The

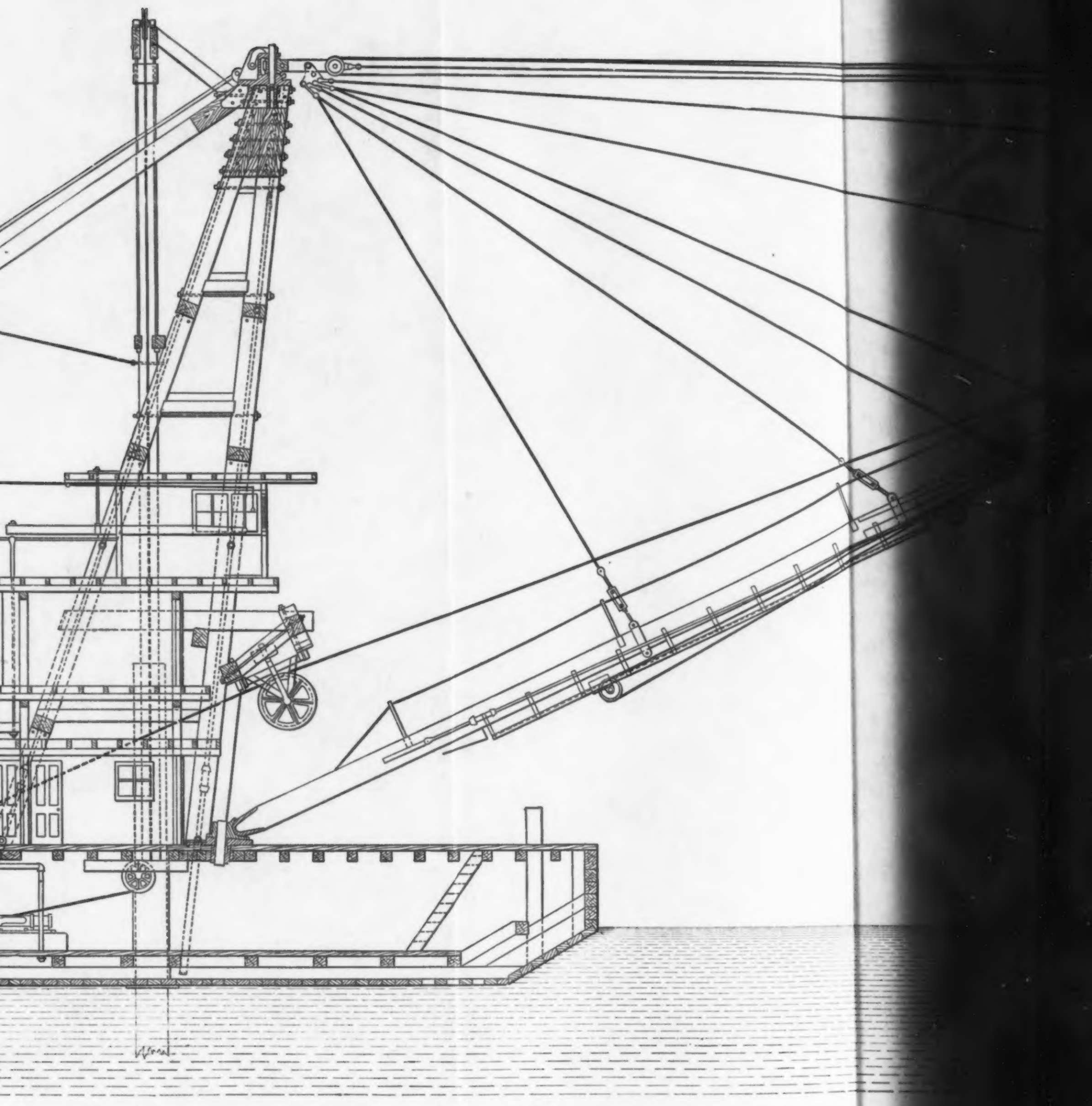
320 000 ft. of Oregon pine lumber and other materials for the hull were bought through the purchasing department of the Southern Pacific Company, and the unusually large timbers required were obtained in Oregon and sent directly to Yuma. In the purchase of both hull material and machinery, time was considered as of the essence of the contracts.

A dredge with a 150-ft. boom, carrying a 5-cu. yd. bucket was decided on, and a hull 120 ft. long, 54 ft. wide, and 11 ft. deep. This width was 2 ft. greater than had ever been built on the Coast, although the tendency is to increase the dimensions, and one is now building in San Francisco, 70 by 140-ft. hull, 205-ft. boom, and 6-cu. yd. bucket. The machinery is a 150-h.p., internally-fired, circular, fire-tube boiler, and a 20 by 24-in. engine on each side. It was decided to build the hull and erect the machinery at Yuma, and float the completed dredge down the river to the intake.

Lumber for the hull began arriving in Yuma late in January, and early in March the company was notified that all the machinery was ready at San Francisco for shipment. Mr. Edinger's connection with the company had ceased soon after the destruction of the Edinger Dam, and Mr. Rockwood had very little confidence in the feasibility of the concrete head-gate plan, or in the desirability or need for the clam-shell dredge, and felt that the great cost thereof would deplete seriously the $200 000 loaned by the Southern Pacific Company. Therefore, practically nothing was done in the matter, and so it came about that the great conflagration in San Francisco, following the earthquake of April 18th, 1906, destroyed the plant of the Golden State and Miners Iron Works, in which all the machinery for this dredge was stored ready for shipment. Fortunately, the damage sustained by the apparatus was not extensive, and by May 15th, 1906, all the machinery had reached Yuma.

Mr. J. W. Brown, a member of the Golden State and Miners Iron Works Corporation, agreed to take charge of building the hull, and reached Yuma about May 1st, bringing with him a complete crew of mechanics and ship builders. Work was hurried, and with such success that the hull was launched about August 15th, the machinery was in place by the end of October, and the dredge weighed anchor and started down the river. At this time the river was getting low and some difficulty was encountered, but on November 26th, 1906, the





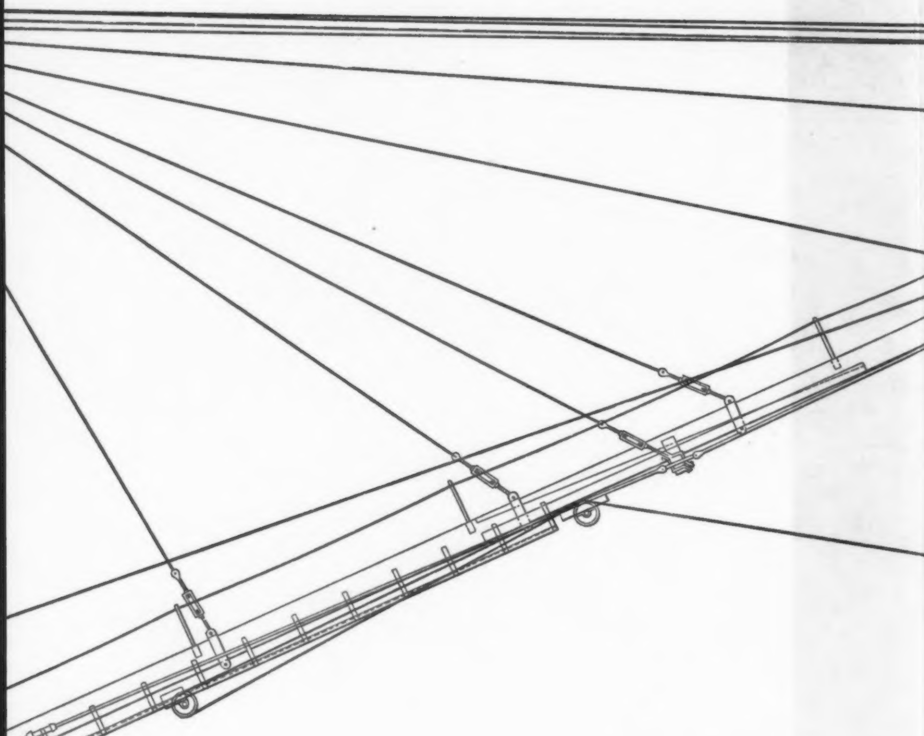
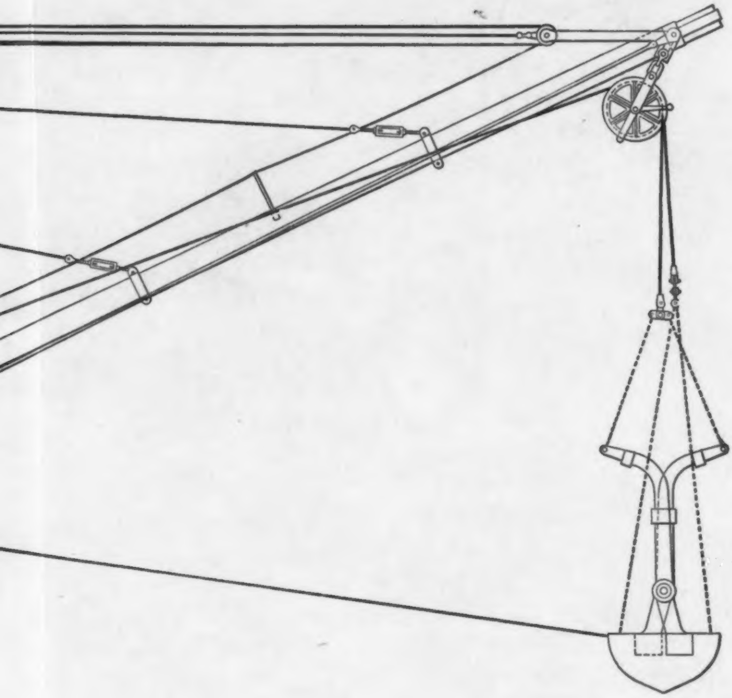


PLATE CXII.
PAPERS, AM. SOC. C. E.
NOVEMBER, 1912.
CORY ON
IRRIGATION AND RIVER CONTROL,
COLORADO RIVER DELTA.



CLAM-SHELL DREDGE "DELTA"

FOR

CALIFORNIA DEVELOPMENT CO.

GOLDEN STATE AND MINERS IRON WORKS, BUILDERS.

SCALE OF FEET

0 5 10 15 20

clam-shell arrived at the mouth of the American intake. The total cost of the dredge, ready to start down the river, was almost $80 000, the cost of the machinery being $34 000, f. o. b. San Francisco. The weight of the craft is about 850 tons.

This dredge, Plate CXII, has been an invaluable piece of machinery to the C. D. Co. Had it been ready for use in August, 1905, the cost of doing the earthwork in the Hind Dam would have been wonderfully reduced. As it was, the dredge, after doing a little work in enlarging the intake above the concrete head-gate, was floated down and cut its way into the Main Canal following the upper Mexican intake. It was engaged on this work when the second break occurred, and continued thereon as though this latter event had not happened. Like the concrete head-gate, it played no part whatsoever in the rediversion of the river.

For the information of those who are not familiar with the results and cost of clam-shell dredge operation, the following data are given:

Operatives.

1 Captain	at $125 to $150	per	month,	and	board.
3 Levermen	" 85	"	"	"	"
2 Firemen	" 60	"	"	"	"
2 Deckhands	" 50	"	"	"	"
1 Cook	" 50	"	"	"	"
1 Blacksmith	" 90	"	"	"	"
1 Roustabout	" 40	"	"	"	"

Three shifts give a total of 22 hours actual work daily. The average time in operation, when proper repair work is done, is 28 days per month. When in good ground, and with side swings averaging 70° on each side, the time per bucketful is 40 sec. The quantity handled (varying according to the material) is from 3 to 8 cu. yd. as ordinary extremes. On the Sacramento River, under good conditions, 150 000 cu. yd. per month are handled.

Monthly expense.

Maintenance and operation	$2 500
Interest on investment at 6%	400
Taxes and insurance	200
Deterioration	700
Sometimes as low as	$3 800

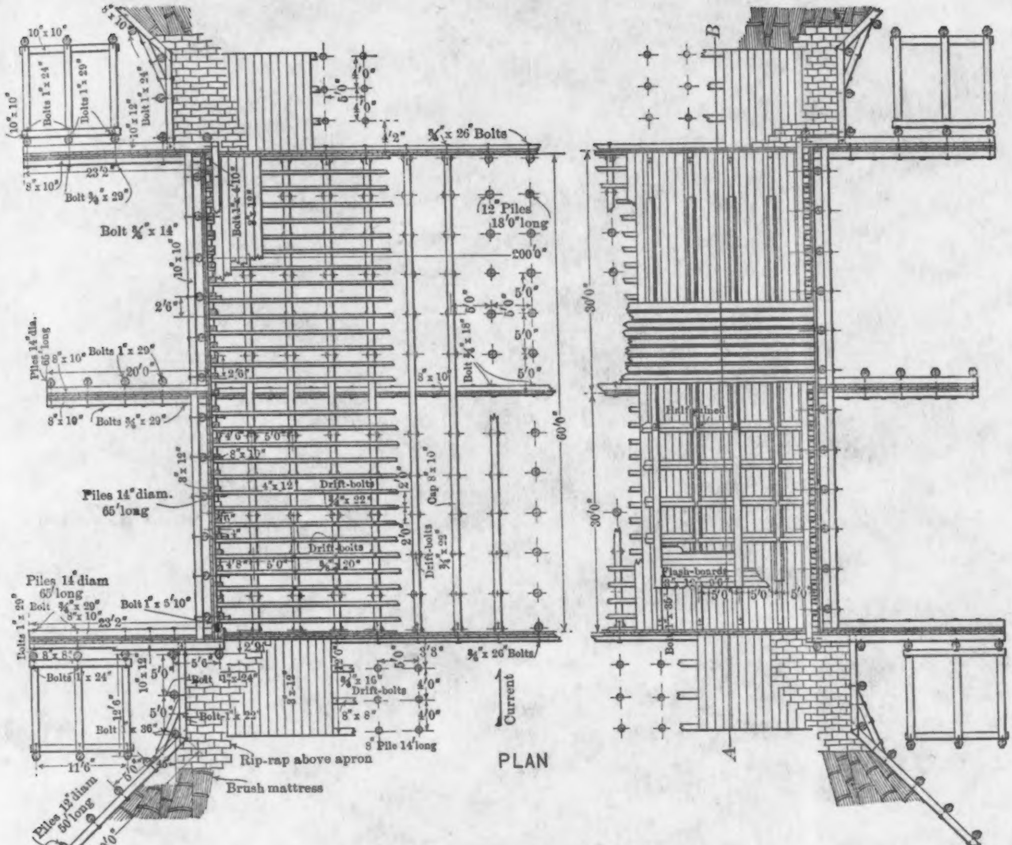
Ordinarily, the monthly expense in Mexico is $5 000. The average cost is 2¾ cents per cu. yd. The average cost of the work done by the *Delta* in Mexico is from 4 to 6 cents per cu. yd.

The Rockwood Head-Gate.

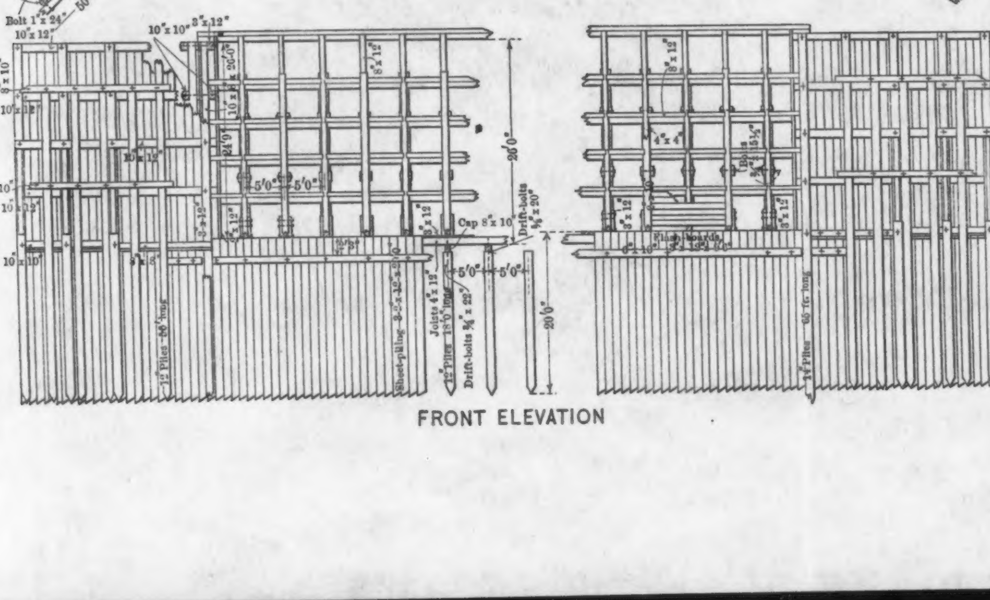
As already explained, it was decided to follow both Mr. Schuyler's and Mr. Rockwood's plans for diverting the river, and so, for the second time, on December 15th, 1905, Mr. Rockwood was authorized to proceed with the construction of a wooden head-gate beside the lower intake. The heavy flood of November 29th and its receding waters had widened the intake from 300 to approximately 600 ft., and, after considering the conditions, it was decided to build the gate directly in the old canal about 200 ft. north of the intake channel, in order to reduce the time and the quantity of excavation required, and to divert the relatively small quantity of water in the old canal around the gate with a by-pass to be dug by the *Alpha*. The gate, started and abandoned three months before, was originally planned for a width of 80 ft.; this was increased to 120 ft. in order to carry a maximum of 9 000 sec-ft. As the gate could not be completed until the spring of 1906, the length was extended to 200 ft. The over-all dimensions, including the wooden aprons, became 240 by 100 ft. The entire space, of course, had to be inclosed in a coffer-dam and the excavation made inside of it. The plans are shown on Plate CXIII.

As far as the writer has ever heard, this is the largest and most daring design ever made for a wooden A-frame, flash-board head-gate. Pile-driving was begun on January 7th, and the gate was completed on April 18th, 1906; the work was rushed day and night for the greater part of the time, and no real difficulties whatever were encountered. As in the case of the concrete head-gate, 4 miles above, the quantity of water seeping into the excavation was surprisingly small. The various items of the cost of this structure were not segregated, so that the details cannot be given, but the grand total expense of the gate proper, exclusive of the by-pass, was approximately $122 500.

The discharge of the river by April 10th, was 32 200 sec-ft., and showed that the annual flood had begun, therefore all idea of attempting to divert the water through the gate by damming the crevasse itself before the summer flood should have been passed, was abandoned.



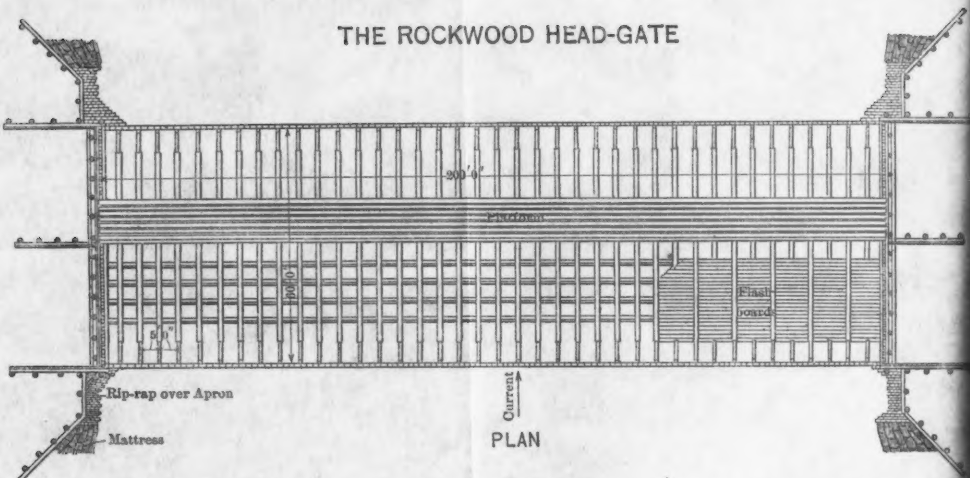
PLAN



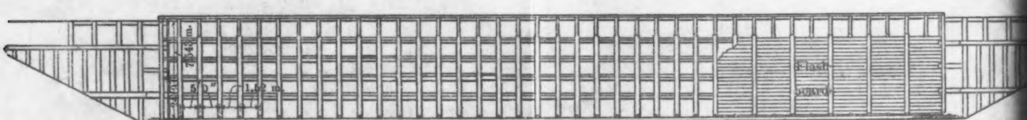
FRONT ELEVATION

PLATE CXIII.
PAPERS, AM. SOC
NOVEMBER, 19
CORY ON
IRRIGATION AND RIVE
COLORADO RIVER

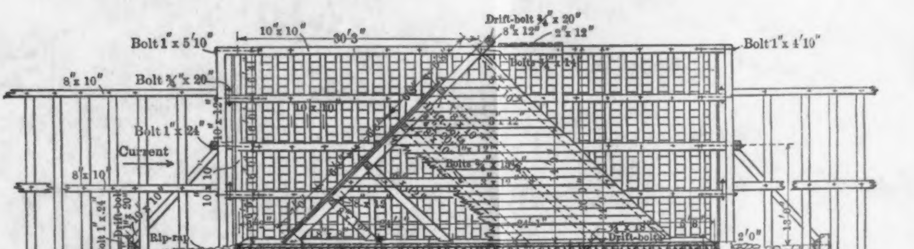
THE ROCKWOOD HEAD-GATE



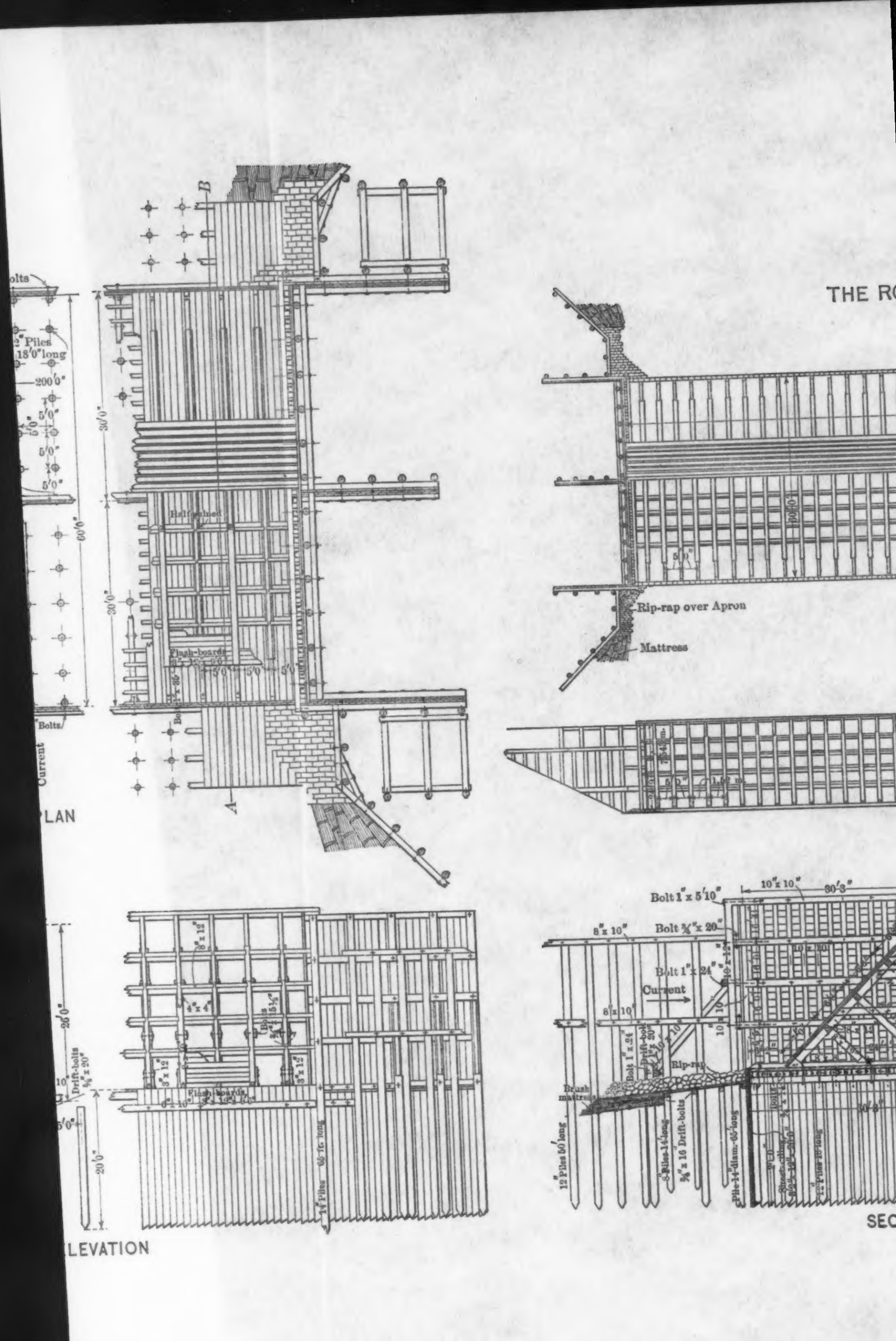
PLAN



FRONT ELEVATION



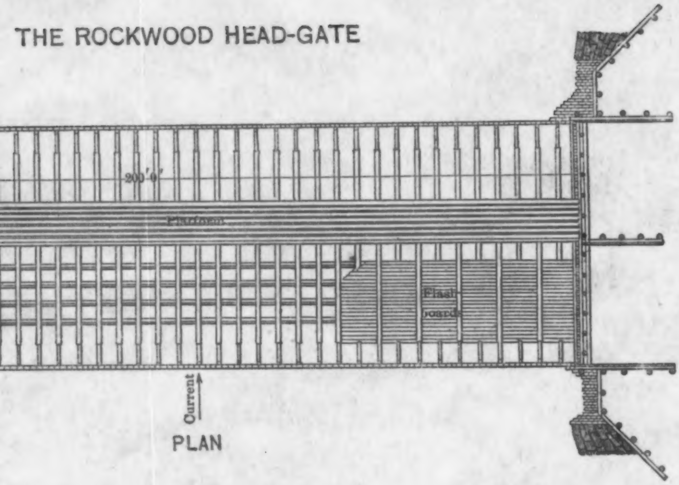
SECTION ON A-B



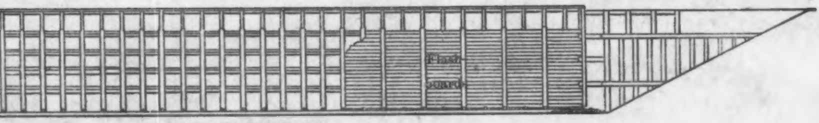
THE RO
PLAN
ELEVATION
SEC
Rip-rap over Apron
Mattress
Current
Flash-boards
Bolt 1" x 5'10"
Bolt ¾" x 20"
Bolt 1" x 24"
Rip-rap
Brush mattress
12" Piles 50' long
Drift-bolts
¾" x 16 Drift-bolts

PLATE CXIII.
PAPERS, AM. SOC. C. E.
NOVEMBER, 1912.
CORY ON
IRRIGATION AND RIVER CONTROL,
COLORADO RIVER DELTA.

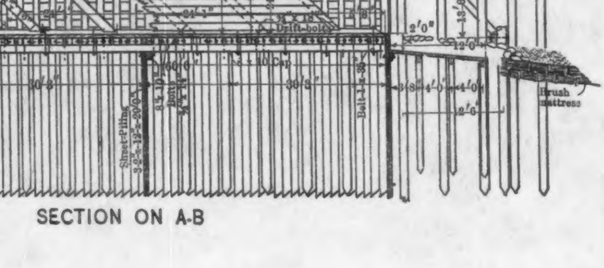
THE ROCKWOOD HEAD-GATE



PLAN



FRONT ELEVATION



SECTION ON A-B

Change in Engineering Staff.—On May 15th, 1905, the writer was transferred from the Southern Pacific Company in San Francisco to the Associated Harriman Lines in Arizona, with the title of Assistant to the President of those properties. About 5 weeks later Mr. Randolph's duties were increased by being put in charge of the C. D. Co. and the Mexican Co., and shortly thereafter the officials of the Southern District of the Southern Pacific Company urged on him the very serious fact that the track beside the Salton Sea would soon be under water, and insisted that aggressive action be taken to close the break on the river. About the middle of July Mr. Randolph sent the writer to the river to confer with Mr. Rockwood, and a day was spent together examining the situation. About August 1st a second trip was made, and after the disastrous flood of November 28th, 1905, a third visit. Toward the end of January Mr. Randolph again sent the writer to the Lower Heading to assist Mr. Rockwood in hurrying the construction of the wooden head-gate. As this work neared completion, Mr. Rockwood suggested that he had found it impossible to handle things in accordance with his own ideas; he believed that the best interests of all concerned pointed to his resignation, and urged that the writer take up the work. After considerable discussion it was agreed that, if Mr. Randolph also desired the arrangement, there would be no objection offered. Shortly afterward these gentlemen met in Los Angeles and agreed to the change, and on April 19th, Mr. Rockwood resigned as Assistant General Manager and Chief Engineer, and was appointed Consulting Engineer, and the writer was appointed General Manager and Chief Engineer. Mr. Rockwood continued to act as Consulting Engineer until October 1st, 1906, when he severed his official connection with the company.

The San Francisco Fire.—On April 18th had occurred the earthquake which resulted in the great San Francisco conflagration, and exaggerated rumors as to the extent of the disaster made it seem certain that the machinery for the *Delta* was utterly destroyed; but that was the least important result, as far as the C. D. Co. was concerned. It appeared that the key city to the Harriman Lines was practically in ruins, and the Southern Pacific Company, as a railroad organization, was very seriously hurt.

Mr. Randolph hurried to San Francisco to join with the other officials in the West in conferring with Mr. Harriman, who had at once

started for the scene. There, in the bustle and confusion of temporary offices, with the ruins of San Francisco still smoking, with the facilities of the road to carry people away from the stricken city taxed to the very utmost, with the wonderful railway system which constituted Mr. Harriman's life work crippled to an unknown extent, and with the financial demands resulting from the disaster impossible to determine, Mr. Randolph succeeded in inducing Mr. Harriman to advance an additional $250 000 for controlling the Colorado River and protecting Imperial Valley. It has always seemed to the writer that this was really the most remarkable thing in the whole chain of extraordinary happenings.

The Situation.—The wooden head-gate was completed, and the upper and lower by-passes connecting it with the break had been fairly well started with the dredges, *Alpha* and *Beta;* the concrete head-gate was well under way; the material for the hull of the *Delta* was in Yuma, and the machinery seriously damaged in San Francisco; the tracks of the Southern Pacific Railroad along the Salton Basin were nearly awash for a considerable length; the annual summer flood of 1906 had begun, and, from the Weather Bureau reports from the drainage basin, would be a very large one; the irrigation system of Imperial Valley was already threatened at several vital points by the excessive quantity of water going down the Alamo channel or Main Canal; and friction between the old C. D. Co. stockholders and the new management had commenced.

No very great degree of reliance could be placed on the wooden head-gate, considering the character of its foundations; and the failure or serious weakness of that structure meant the failure and abandonment of the Rockwood plan for re-diversion. The difficulties of the Concrete Gate Plan, under the most favorable circumstances, became more apparent with further investigation, and were very greatly accentuated by the delay in getting the *Delta* into commission. The probability of the withdrawal of financial support at any time through the discouragement of the Southern Pacific officials as to the ultimate success of the work was a serious factor. Transportation facilities from Yuma were very inadequate, consisting of the steamers, *Searchlight, St. Vallier, Cochan,* and the barge, *Silas J. Lewis,* all of sufficiently light draft to navigate through the shoals and sand bars of the Colorado. There were large quantities of willow brush suitable for fas-

cines and mattress work near the break, but no timber suitable for piling. The nearest point where piles and heavy timber were obtainable was Los Angeles; from there they came by rail to Yuma, from which point they could be floated down the river only at considerable risk, so that it was cheaper to load them on barges and bring them down with steamboats.

Experience thus far had indicated the practical impossibility of closing the break with a piling-brush-sand bag barrier dam, and there were no quarries for many miles either west or east along the railroad, and none, of course, available except with railroad facilities for loading and transportation. Further, rock would require to be transferred to barges at Yuma and be brought thence by river to the scene of operations.

Practically every engineer—and they included many of established national and international reputation—who had visited the break considered a rock fill barrier dam as entirely unworthy of consideration, for two reasons:

First, it was believed that rock would sink into the soft alluvial silt bottom and keep on going down indefinitely, even if more and more slowly. Old river men quoted numerous instances of wrecked river craft. They cited a dredge, bought a few years before by the C. D. Co., which had sunk on its way from Yuma to the upper intake, gradually settling entirely out of sight in a few months. The consensus of opinion, therefore, was that any rock fill would certainly settle out of sight unless built on a very strong brush mattress foundation, and the probabilities were great that such a mattress would break under the load and fail of its purpose.

The second vital objection urged against a rock fill barrier dam was that the water going over it while building would dislodge some portion of the fill or some one rock at the top, thereby increasing the overpour at this point, which would dislodge more rock and in this way quickly result in a breach which could not be closed.

It was thought that these considerations not only quite precluded the idea of a barrier dam, should the wooden gate fail, but rendered very doubtful the construction of a diversion dam or obstruction in the channel opposite the gate which would cause a difference in head, above and below it, great enough to throw all the water through the by-pass and gate. This head was variously estimated at from 3 to 6 ft.

—the head on the finished dam would be about 15 ft. at low-water stage.

On one point there seemed to be accord, namely, that the situation was a desperate one and without engineering parallel, and that there seemed to be little more than a fighting chance of controlling the river. No two of the nearly fifty eminent engineers, who visited the scene and examined into the situation more or less carefully, agreed on any one plan as offering the greatest chances of success, but pointed out fundamental weaknesses in practically all other methods suggested. This feature was so marked that when the writer suggested to President Randolph that the immensity of the interests dependent for their safety on the re-diversion of the river seemed to render advisable a Board of Engineers, he answered that he would regard 100 ft. of good strong brush mattress in place on the river's bottom as more valuable than the report of any Board of Engineers which could be gotten together.

The immediate menace, however, was from the summer flood in passing through the Imperial Valley to the Salton Sea. The Weather Bureau's reports from the upper drainage basin then indicated a very great total discharge, and a peak perhaps as high as 100 000 sec-ft. The crevasse had now enlarged, and the old channel below had filled up, so that practically all this water—several times as much as had ever yet entered the valley—must go the new way.

Summer Flood of 1906.—Plate CIX shows that, compared with recent floods, the summer flood of 1906 was very large, although it has been greatly exceeded since then, notably in 1907 and 1909. The increased fall down the Alamo River channel resulted by August 1st in lowering the river at the diversion point approximately 4 ft., but it silted up as the flood receded, leaving a net lowering of between 2 and 3 ft. (Fig. 3.)

It widened the break from 600 to almost 2 700 ft., and rendered far more expensive, in time, equipment, and money, the task of putting the wooden head-gate into commission. The most important effect, however, was the danger it caused in various ways in the Imperial Valley proper.

Such a vast quantity of water going down the Alamo channel was, of course, never contemplated in designing the new waste-gates near Sharp's Heading discharging down the Alamo River (built June to

August, 1905), and at Station 134 on the Central Main, and they were taxed to their absolute limits. So much passed the Alamo Waste-gate that it caused a recession of the grade in that channel below, so that the structure was, figuratively speaking, on stilts. Twice the chute below the structure had been extended, the last time in February and March, 1906, when the equipment was removed just as the water began to go over the top of the gates.

By a peculiar and most fortunate coincidence, when the Alamo Waste-gate was discharging approximately 3 500 sec-ft. and Sharp's and the Encina Head-gates were being utilized to the capacity of the canals below them, the water in the Alamo above this point spread over-bank for miles, going to the west and south sufficiently deep to save the situation. Thus it happened that when the peak of the flood was reached, and approximately 75 000 sec-ft. were going down the Alamo channel toward the Salton Sea, all but about 5 000 sec-ft. were going overbank to the south and west. Had not this most fortunate condition existed, the Imperial Valley irrigation system would early have been broken into the deep channel of the Alamo below the waste-gate, and at once cut the water out of every canal.

Most of this overbank flow to the south and west collected in the various sloughs and low lands, particularly Beltran's and Garza's Sloughs, and flowed into the New River. The small channel of this watercourse was overtopped, of course, and the water spread out, just south of the Boundary Line near Calexico, for a maximum width of about 10 miles. Some of the water overtopped the divide of the delta cone, gained the Paredones channel, and thence ultimately reached the Gulf.

The most critical points were where the New River channel crossed the Boundary Line, and a little farther down along the Central Main. At Calexico and Mexicali this broad sheet of water rose until it covered the ground about 4 ft. in depth. (Figs. 2 and 3, Plate CXIV.) The danger was not appreciated in time to throw levees to the west of the railroad track and thus protect that property. The disposition of the towns and the railroads was to wait for the C. D. Co. to build protective levees, in spite of that company's announced intention of doing nothing of the sort.* When the situation was finally realized, about 5

* This was because the company's attorney advised that it was not responsible legally for damages caused in the United States by operations of the Mexican Company in Mexico, and to avoid carefully any action which might be considered as an admission of responsibility by the company.

miles of levee—maximum height 5 ft.—encircling the two towns and connecting at the north and east with higher ground, was hurriedly built. Strong winds blow in the spring for two and three days at a time, and when such storms swept over a wide stretch, even though the ground had a considerable quantity of brush, waves were caused which made the maintenance of these levees at times very critical. They were held successfully, however, until the recession of the New River grade made them no longer necessary.

Along the Central Main, from near the branch railroad crossing west to beyond the "Five Gates" (where the canal turns to the north), the water rose so high during the last days of February that it overtopped the south bank of the canal, and only by the most desperate work was it prevented from overtopping the north bank and sending water northeastward across the country to the Mesquite Lake Basin and the Alamo channel. Had this occurred, the Town of Imperial would have been most seriously threatened, perhaps destroyed, and the New River and Alamo chasms would have been joined by a third one, about 25 miles long, diagonally across the valley northeast and southwest. The C. D. Co. then greatly strengthened this north bank and raised it 4 ft. for a distance of nearly 3 miles. When the situation was most threatening the citizens of Calexico and Mexicali were called out to help hold the levees, while the people of Imperial rushed down to aid in the fight along the Central Main.

Both the Alamo and New River channels cut back, owing to the large quantity of water flowing in them, and the Salton Sea began to rise at the rate of approximately 7 in. per day. The Southern Pacific main line there was being shifted from time to time, by means of "shooflies." Along the branch line from Imperial Junction to Calexico the trouble at the crossing of the Alamo channel was far greater than should have been permitted. At no time was more than 3 500 sec-ft. going down the Alamo, yet this small quantity was permitted to eat away approximately 300 acres of land, in a semi-circular form, from the right bank of the channel where it is crossed by the branch railroad into the valley, and caused the railroad to "shoofly" its tracks five times. The alluvial soil of the Imperial Valley is very easily eroded, especially on the concave side of river bends, but it should have been possible to control at reasonable cost a stream of 3 500 sec-ft., with a velocity never exceeding 7 ft. per sec.

FIG. 1.—ALAMO WASTE-GATE, NOVEMBER 17TH, 1906. ABOUT 30 FEET HEAD AGAINST GATE.

FIG. 2.—PORTION OF 4-MILE LEVEE PROTECTING CALEXICO AND MEXICALI, IN FLOOD OF JUNE, 1906.

FIG. 3.—OVERFLOW AGAINST WEST AND SOUTH BANKS OF MAIN CANAL NEAR FIVE GATES, 2½ MILES NORTHWEST OF CALEXICO.

The Inter-California Railroad from Calexico toward Yuma had been constructed as far as Cocopah and practically all of this was under water. The Holtville Interurban Railroad, crossing the Alamo River, was cut out from time to time, the channel at that point being lowered more than 30 ft. This caused serious trouble with the discharge pipe of the Holton Power Company, the plant being left rather high, and considerable work was required to keep it from being undermined by side cutting. The head available, however, was increased by 30 ft.

When the grade of New River had receded to a point about 3 miles above the International Boundary Line, a large area of adobe formation was encountered, and the fingers of the stream began to eat away in various directions and threatened to tear up the country throughout that region in a frightful way. The rate of recession was also greatly slackened. Long before the peak of the flood had been reached, it was evident that the situation along the Central Main and at Calexico was very serious and must become much more so, until grade recession might give relief. It was decided, therefore, to use dynamite liberally in an endeavor to localize the New River's grade recession and to facilitate its progress.

From observations and soil and other data at the time available the probability seemed slight of such recession extending more than 6 or 7 miles beyond Calexico, or far enough to endanger lowering the water surface in the Alamo above the controlling works and so menacing the water supply of the valley before the summer flood of 1908. It was known that very large areas of adobe soil existed in the Garza's and Beltran's Slough country, so that the cutting there would be very much slower. There thus seemed to be considerable leeway, while the strain on the irrigation system of the valley was so severe at several critical points that it was utter nonsense to think it could be held through another flood season.

In this dynamiting, from eight to sixteen ½-lb. sticks of dynamite were tied in a bundle about a fulminating cap connected with from 8 to 12 in. of water-proof fuse. The fuse was then lit and the bundle tossed into the water. A little practice and careful observation enabled one to become quite proficient in estimating how far the bundle of dynamite would be washed down stream by the current before the cap exploded the charge, and in placing the charge to get maximum

results. Undoubtedly, the course of the grade recession was considerably checked and bad erosion somewhat mitigated by this work, but it is very doubtful whether the time of the grade recession's passing Calexico and Mexicali was markedly accelerated.

When this occurred the results were spectacular in the extreme, the rate of cutting back at this point being fairly uniform at 1 ft. per min. The side cutting of the east bank of the wide, deep barranca for several days threatened Calexico, and carried away a considerable part of Mexicali, including the railroad station, brick hotel, and a number of smaller buildings. The actual damage sustained was about $15 000 in Calexico and $75 000 in Mexicali.

For a short distance past Mexicali the cutting back followed the borrow-pits of the Inter-California Railroad, utterly destroying it and carrying away much of the track and trestle material. About a mile out of town, the grade rose slightly above the flood-waters, but farther on, for several miles again, the roadbed was practically destroyed, although no more track material was lost.

These flood-waters covered about 6 000 acres of cultivated farms, of course, utterly ruining the crops. Greater damage, however, occurred as the grade receded and the water rushed from each side toward the newly-made channel, resulting in cutting back fingers or side cañons from the main stream to distances and depths depending on the length of time required to drain off the contributory water. Some of these side cañons extended back from 2 000 to 2 500 ft. It thus happened that about 3 000 acres of improved and 10 000 acres of unimproved land were eroded to such an extent as to be practically ruined for agricultural or any other purposes. Of this area, about 7 000 acres were public land. The area occupied by the New and Alamo channels themselves was increased by about 7 000 acres.

The greatest damage in the Imperial Valley proper, however, was caused by the destruction of the flumes in the West Side Main over New River in Mexico and the Central Main over New River northwest of Imperial, leaving Mutual Water Companies 6 and 8 without water until January, 1908. These two districts contained about 30 000 acres in actual cultivation, and were rendered practically uninhabitable and absolutely waterless for about 1½ years.

Except as noted, agricultural operations in the valley were facilitated by the flood, there being at all times plenty of water in the

canals. Prospective settlers, of course, were kept away almost entirely, but the inhabitants of the valley displayed a remarkable confidence that the trouble would be overcome, and business was not affected very seriously. Indeed, during these very times, the new and independent town site of El Centro was the scene of really wonderful building activity, and the Holton Power Company, directly and indirectly, practically doubled its plant and holdings in the valley.

The effect of this flood, in a geological way, was of extraordinary interest and very spectacular. In 9 months the runaway waters of the Colorado had eroded from the New and Alamo River channels and carried down into the Salton Sea a yardage almost four times as great as that of the entire Panama Canal. The combined length of the channels cut out was almost 43 miles, the average width being 1 000 ft., and the depth 50 ft. To this total of from 400 000 000 to 450 000 000 cu. yd. must be added almost 10% more for side cañons, surface land erosions, etc. Very rarely, if ever before, has it been possible to see a geological agency effect in a few months a change which usually requires centuries.

Preparation for Diversion Work.

All measures to prevent avoidable damage to the irrigation system in the valley from the flood-waters having been arranged, operations were resumed on the river. The break opposite the wooden head-gate had been widened during the flood from 600 ft. to more than ½ mile, and necessitated work on a far larger scale than had ever been suggested. The opinions to the contrary notwithstanding, the ability to get rock in large quantities and rapidly seemed to the writer to be so essential, and it was so obvious that much better transportation facilities were required, that it was decided to build a branch railroad from the Southern Pacific main line at a point 7 miles west of Yuma (now known as Hanlon's Junction) to the break.

The located line of the Inter-California Railroad, construction of which had been stopped by the overflow waters at Cocopah, ran only a few hundred yards west of the wooden head-gate and 150 ft. west of the concrete head-gate. This Inter-California Railroad is a Mexican subsidiary of the Southern Pacific Company, and it was not difficult to arrange a change in its alignment to cross the Alamo where the best location for the diversion dam could be found and to build

at once that portion from the break north to the concrete head-gate. Thence northward the permanent alignment was expensive and would require considerable time to construct, therefore it was decided to make a temporary connection of about 6 000 ft. from Hanlon's Junction to the concrete head-gate. It was arranged that the Southern Pacific should build the entire branch line and charge the total cost, on a force account basis, to the C. D. Co., and when later, if ever, the Inter-California Railroad should be completed, all that portion of the branch which could be incorporated with the permanent alignment of the road would be taken over by it at such a figure as it would cost at that time. The stretch from Hanlon's Junction to the western line of the lands of the C. D. Co. is in the Yuma Indian Reservation, and, according to the rules and regulation of the Interior Department, it would have taken some time to acquire a right of way for this portion. As it was feared that special permission might not be quickly obtainable, nothing whatever was said, but the line was simply built. Such a course was deemed justifiable, considering the gravity of the situation, the necessity for haste, and the very small discretionary powers given to Government officials in such cases. As soon as the existence of this track was no longer absolutely vital, permission was requested in the usual way and in due course was obtained. Construction of this branch line was begun on July 1st, and on August 15th the first train load of materials passed over it to the Lower Heading.

Quarry.—The granite point of rock on which the concrete head-gate was founded seemed favorable for quickly developing a quarry where a large quantity of rock might be obtained, and instructions were given to do the best possible with it. The rock is a second-class granite, and, before the first closing was completed, a quarry had been developed with a 600-ft. face averaging 40 ft. in height. The development of this quarry and track room for outfit cars, locomotives, etc., called for the building of a large yard of sidings and spurs. This quarry was entirely on C. D. Co. land—that bought from Hall Hanlon at the very beginning.

Clay Pit.—Between the quarry and the Boundary Line, and about ¼ mile west of the branch railroad, there was an opportunity to develop rapidly a clay pit. Advantage was taken of this, and by the time the first closing was completed, there was a steam shovel face, 600

ft. long and averaging 60 ft. in height. The clay in this bed is rather hard and requires some blasting, but it melts down in water, and when mixed in about equal proportions with the cement gravel from the Mammoth gravel pit makes a very impervious material for dam construction.

The Mammoth Gravel Pit.—This pit is on the Southern Pacific Railroad 41.08 miles west of Hanlon's Junction. It had been thoroughly developed at that time and had been used for ballasting the main line for more than 100 miles in each direction. It is the property of the railroad, and the material obtained there is fairly high in clay, the result being essentially a cementing gravel, which makes the surface of the track almost impervious.

Other Quarries Available.—At Declez, a point on the Southern Pacific Railroad 195 miles west of Hanlon's Junction and 49 miles east of Los Angeles, there is a large, well-equipped quarry of very good granite, from which material for the construction of the breakwater at San Pedro Harbor, 19 miles southwest of Los Angeles, is obtained. The output of this quarry is very large, the rock running up to 12 tons.

Near Ogilby, 7 miles west, a large area is covered with lava "niggerhead" rock, essentially one- or two-man size, which had been in part denuded to furnish rip-rap around the railroad bridge over the Colorado at Yuma. The tracks, however, had been torn up, and no stone had been obtained therefrom in years.

At Tacna, 52 miles east of Hanlon's Junction, there was a quarry formerly used by the railroad but abandoned because the rock therefrom was small and of poor quality.

At Patagonia, on the branch line south from Bunson toward Nogales, and 370 miles east of Hanlon's Junction, there was a well-equipped quarry controlled by the Southern Pacific. Its output was a reddish limestone, considerably smaller than that at Declez, but yet frequently turning out 10-ton rock.

These four sources of supply constituted the utmost possibilities, aside from the quarry which might be developed at Andrade.*

Brush.—By no means all the area contiguous to the Colorado is covered with willow brush, but it occurs in spots, often of very large extent. Such areas on the west bank of the river near the Edinger

* Andrade is the name of the Inter-California railroad station on the American side of the Boundary Line, Algodones being on the Mexican side.

Dam had been cleared away, and west of the old Main Canal there was an old shallow lake which, though now drained, was practically barren. All brush, therefore, had to be obtained from the south side of the break, and with an average wagon haul of about 1 mile. The growths, ranging from 6 to 18 ft. in height, were ideal for fascines and mattress work. Main and branch roads were cut by Indian labor in order to get this material to the front rapidly.

Dredges.—The dipper dredge, *Alpha,* and the suction dredge, *Beta,* were in reasonably good condition, but the former could not be used in the sand bar left exposed in the bottom of the break when the waters receded, because the material slipped down to such a flat slope that it would have imprisoned the craft. After doing all it could in the by-pass and more solid ground, it was started to deepening the old Main Canal toward Algodones. Dams were built behind it from time to time, and water was pumped into the canal at the upper intake to keep the machine afloat. The quantity of water required indicated a surprisingly small seepage loss from this old canal into the surrounding country, and this is in accord with the unexpected experience with the coffer-dams of the wooden and concrete head-gates.

Steamers and Barges.—During the latter part of 1905 the Mexican Co. purchased the steamer, *Searchlight,* 91 ft. long, 18 ft. wide, and drawing, without load, 18 in. of water. It had a barge, about 55 ft. long and 25 ft. wide, on which most of its load was carried. The steamer, *Cochan,* 135 ft. long and 31 ft. wide, the largest on the river, belonged to Yuma parties, and as it had been leased by J. G. White and Company for hauling materials and supplies to the Laguna Weir, it was not available. There was another steamer on the river, the *St. Valliers,* 75 ft. long, which was a little smaller than the *Searchlight* and in very poor condition. In addition to these there was the barge *Silas J. Lewis,* 115 ft. long and 35 ft. wide, which was fitted with a donkey engine with which it was pulled up stream. This barge was rented for $15 per day, and its deck was cleared for mattress weaving.

Grading Outfits.—The Southern District of the Southern Pacific Company—from Santa Barbara and Fresno, Cal., to El Paso, Tex.—has enough reconstruction and betterment work to keep two or three grading contractors' outfits at work except during the very hot season. An arrangement was made with one of these, Shattuck and Desmond,

to supply an outfit on the force account schedule paid by the railroad, with provisions for the payment of all duties and for all stock dying from heat. This firm secured, fed, and boarded its own laborers. Inasmuch as there was no very definite plan as to the work which would be required, no contracts were feasible, hence the force account arrangement. At one time about 800 head of this firm's stock, with complete camp equipment, Fresno scrapers, plows, etc., were on the work.

Materials and Stores.—Arrangements were made with the Southern Pacific for equipment, materials, and stores on the basis of cost plus 10%, and for freight charges of 0.5 cent per ton-mile, until the provisions of the Interstate Commerce Commission prohibiting such freight arrangement went into effect. Two steam shovels were brought in for quarry work and one for the clay pit. Complete work trains were requisitioned from time to time until a maximum of ten was reached. A roundhouse foreman and an assistant master car repairer were sent by the railroad company, and temporary, but effective, plants were installed at Andrade. Three carloads of repair parts and stores for engine, car and air-brake repairs were sent out, used from, and returned when the work ended. All requisition blanks, rules, and other organization methods of the railroad were continued.

When the Southern Pacific built the Lucin Cut-off, consisting of a long trestle bridge and an immense fill across Great Salt Lake, in Utah, there were bought a large number of steel side-dump cars, of 45 cu. yd. capacity, locally known as "battleships," weighing approximately 20 tons, and having a capacity of 100 000 lb. with a permissible 10% overload. These cars were frequently loaded to 125 000 lb. on this work, as the trip between the Andrade quarry and the break did not exceed 4 miles. At first 80 of these cars were secured, and more and more were sent until about 300 were finally in service. Such a quantity of railroad equipment necessitated rather extensive terminal facilities, and these were provided on the American side of the line because of the customs regulations of the Mexican Government.

The railroad from Hanlon's Junction to the Lower Heading, the quarry, clay pit, steam shovels, etc., were under Mr. Eulogio Carrillo, Assistant Engineer of the Southern Pacific Construction Department, from June 1st, 1906, to July 21st, 1907, as a superintendent of the C. D. Co., from which he received his salary, the railroad giving him

leave of absence for that period. All the men under his direction, however, were carried on the Southern Pacific payrolls, and bills were rendered later by that corporation to cover this expenditure.

There were two reasons for having the railroad company supply so great a quantity of labor, equipment, materials, and supplies. First, it afforded an opportunity to assemble quickly a thoroughly organized and efficient force of men; the advantage of obtaining materials and supplies at low prices by the purchasing department of the Harriman systems; immediate shipment of repair parts not kept on hand, thus reducing delays to the minimum; and the ability to increase or decrease rapidly the force and equipment without confusion. The second reason was that no immediate cash was required, and as bills of all kinds were not usually presented and approved in less than about 6 months, approximately 3% in interest was saved. All bills were rendered at actual cost plus 10%, which thus meant really cost plus 7%—a very low figure for superintendence, etc.

Whenever any train, equipment, or men left the main line and came on the branch line they reported to and were under the jurisdiction of Mr. Carrillo, who in turn reported to and was under the sole jurisdiction of the writer. In this way no misunderstanding arose, and the entire force obeyed instructions issued as quickly and fully as though there were absolutely no connection between them and the Southern Pacific.

Storehouse at Lower Heading.—Duty had to be paid on everything taken into Mexico, but, nevertheless, a very complete storehouse of repair parts, small tools, etc., was established at the Lower Heading. No requisition system was put in, however, because it was felt that the losses which would thus occur would amount to much less than the delay due to any form of red tape, whatsoever. Everything received was charged to the work, and at its closing down an inventory was made and the work was credited with the value of the material left.

Climatic Conditions.—From about June 1st to the middle of September or October 1st, the temperature of this region is so high that until 10 years ago it was not considered advisable to continue large construction work during that season. There can be no doubt that ordinary labor is only from one-third to two-thirds as efficient in such heat, and during this particular year the general average seemed to be about one-half. There is little wind during this period, and the

humidity is ordinarily very low, though occasionally it is quite high for periods of two or three days.

Mosquitoes are frequently a terrible pest, very often driving even cattle out of regions near stagnant water. There is relatively little vegetation about Andrade, and at the Lower Heading a large camp compound was entirely cleared and the stagnant pools in the vicinity drained at a slight cost, so that the mosquitoes, while annoying, were by no means serious.

Brush and arrow weed growths are so dense that white men, no matter how well acclimated, cannot work very hard in cutting them down. Men from the central part of Mexico were imported, but they could stand it little better. Indian labor is the only kind for that sort of work.

Labor Conditions.—The work of rehabilitating San Francisco after its disastrous conflagration drew there an immense amount of shifting labor. To the south Los Angeles was growing in every direction. The Harriman Lines, under President Randolph, was employing large numbers of men constructing the West Coast Railroad from Guaymas toward Mazatlan and Guadalajara. Much betterment work was in progress on the lines from Los Angeles to El Paso, and large forces were required for building "shooflies" and shifting track along the Salton Sea. J. G. White and Company were rushing work on the Laguna Weir, and the Reclamation Service was building the Roosevelt Dam near Phœnix. Thus the labor situation in California as a whole, and in this part of California in particular, was acute. The immigration laws of the United States prevented the importation of Mexicans, except in a very small way, but here the work was in Mexico. It was decided, therefore, to obtain laborers from Central Mexico, ship them from El Paso to Yuma in bond, and back into Mexico at the Lower Heading. Arrangements were made with the Labor Agent for the Southern Pacific, Southern District, Mr. Ben Heney, of Tucson, to ship 500 men. This plan was an utter failure, for two reasons. The Mexican officials did their best to prevent Mr. Heney's agents from getting men started, and the 75 men who arrived were unable to stand the climate.

Attention was then turned toward getting Indians in large numbers, and arrangements were made with Mr. C. E. Dagenette, Indian Outing Agent, with the result that, by the time work was in full

swing, practically all the men, women, and children of six Indian tribes were on the work—the Pimas, Papagoes, Maricopas, and Yumas, from Arizona; and the Cocopahs and Diegueños, from Mexico. These six tribes fraternized and got along together without any difficulties whatever, and constituted a separate camp of about 2 000 people. About 400 workmen could be depended on from this collection. They were paid 20 cents an hour, and every 9 men received in addition one man's pay to go to a squaw for cooking their food. The Indians bought their own supplies, and to avoid duty built their camps on the Arizona bank, crossing the dry channel below the break to and from work.

Indian labor was very satisfactory, and, indeed, just what other arrangement could have been made is very problematical. Under intelligent foremen who understand their peculiarities, chief of which is lack of assurance and consequent timidity in going ahead with work, they are quite satisfactory. They must be paid weekly, and very few can ever be induced to work on Sunday or to put in overtime, regardless of how critical the stage of work may be when the whistle blows.

Very fortunately, indeed, an unexpectedly large amount of floating labor came in from every part of the United States, men who are attracted to any work which has achieved notoriety for any reason. Once on the ground these men did not work any great length of time. A work train ran into Yuma every night for provisions and supplies, returning early in the morning, and it always carried a considerable number of cheerful capitalists out and sadder and wiser men in. Yuma at that time was "wide open," with all sorts of lures which few of these floaters could resist. To what extent the work would have suffered had Yuma then been a closed town, as it is now, is a question.

The general wages paid were:

Pile-driver foreman	50	cents per hour.
Pile-driver donkey runner	$43\frac{3}{4}$	" " "
Good pile-driver helpers	$31\frac{1}{4}$ to $37\frac{1}{2}$	" " "
Ordinary labor	$27\frac{1}{2}$ to 30	" " "

Work from 8 to 10 hours per day.

Board deduction, $22.50 per month.

Commissary and Camp Plans.—The usual outfit cars were provided for all men carried on the rolls of the railroad, and many were boarded

in the dining cars, which were a part of Mr. Carrillo's permanent construction outfit. The remainder of the men were boarded by Mr. M. C. Threlkeld, of San Francisco, who had and still has a contract with the railroad to board all gangs engaged in maintenance of way and betterment work on its lines. Mr. Threlkeld took an essentially similar contract for feeding the white laborers of the C. D. Co., the first contract being for 25 cents per meal in the United States and 40 cents in Mexico, the contractor to pay all customs duties on material and supplies. After the second break, and when the work was continued at President Roosevelt's request, it was deemed probable that the Mexican Government would refund duties on provisions thereafter, so that the contract was changed on January 1st, 1907, to 25 cents per meal and the Mexican Co., to pay the duties. This contract covered meals for all white laborers, including men on dredges, on the steamer *Searchlight*, etc., and gave Mr. Threlkeld the exclusive selling of clothing, tobacco, notions, etc., to the laborers. The Indians bought relatively little from him, however, preferring to deal with Yuma merchants with whom the local Indians were very well acquainted.

Excellent board for the men was insisted on and furnished. It was believed that good board, especially with lots of fresh vegetables, would be a large factor in keeping men on the work, and this was found to be the case. Large numbers of mosquitoes were feared, in spite of precautions taken, so bunk houses were built, with brush ramada roofs, and carefully and effectively screened all round. These precautions were not exactly necessary, but were nevertheless well worth their cost.

Policing of Camps.—The many different classes of laborers on the same job and under Mexican laws made it essential to have effective police arrangements, and bar liquor from the camp absolutely. The Yuma Indian Reservation extends to the line, and, in addition was then and until 1908, a part of San Diego County, and a "dry" region. Across the river in Arizona is "wet," but the United States laws against selling liquor to Indians are rigorously enforced. In Lower California, however, the idea of liquor control has not even germinated, and it was necessary to promise to prevent American Indians from getting liquor in Mexico before permission could be obtained to take them out of the United States—and this was quite proper. Accordingly, arrangements were made with the Mexican authorities to put the entire region under martial law, and send a force of rurales with a military

commandant at their head to police the camps. This proved extremely efficient and satisfactory, and there was absolutely no disorder at any time.

Customs and Duties.—Except for the operations of the C. D. Co., there was no development in Mexico along the river, therefore, until 1908, the nearest custom house in Lower California was at Mexicali. A garrita was maintained at Algodones, however, where material going down the river to land in Mexico was passed. During the construction of the Edinger Dam, all camps and supplies were kept on Disaster Island in the middle of the river, so that there were no customs charges. When the construction of the wooden head-gate was begun, endeavors were made to get the Mexican Government to establish a customs office at Algodones temporarily, but without success. Accordingly, all bills of material to be passed had to be sent to the custom house in Mexicali; there the charges were assessed, and the manifest was returned to Algodones before the goods could be taken over, which was very cumbersome and slow.

Another method of getting goods across the line was taken advantage of, namely, by boletas. The Mexican Government permits each individual, on payment of duties, daily to take across $20 (Mexican) worth of dutiable stuff without manifest, and the authorities agreed to permit goods to be passed at the Algodones garrita by this boleta method, having individual employees of the company sign the boletas. In this way emergency stuff was passed.

Under the concession of the Mexican Co., machinery and materials for permanent construction was to be admitted without duty, but the intention of this provision was plainly for the company to make out a list of what would be required once for all, and that such freedom from duties would apply to the original entry of the machinery and material, and not to subsequent repair parts, etc. Obviously, it did not contemplate the refund of customs charges in such a case as closing the crevasse. Nevertheless, it seemed probable that the customs charges for material and supplies other than provisions would be refunded, because the Mexican Government itself was vitally interested in stopping the break. Tentative negotiations toward this end were started, but the procedure for securing such permission is a long one, and it was advised that the work be prosecuted and the request for refund made after its completion. It was also made plain that no refund

would be given for duties on provisions, as it was impossible to determine that the provisions passed were all actually used on the work. When the work was completed a request for a refund was made, and, on President Diaz's recommendation, the National Congress, by vote, refunded approximately 75% of all duties paid, amounting to more than \$40 000.

The chief objection, therefore, was the red tape involved in passing goods, and the delays which followed any slight technical mistake in classification. As an illustration: an inspector investigated the customs transactions of the period about a year later, and assessed a fine against the company for \$3 000 for utilizing the boleta method of passing emergency materials and supplies. On proper presentation of the facts, however, this fine was remitted. Stock with harness and grading equipment was permitted to be passed into Mexico under bond for a period of 6 months, as also was machinery, which provision assisted very greatly in the work.

All payrolls, time checks, receipts, and legal papers require stamps to be affixed and cancelled, inspectors from time to time visiting all corporations and checking the books. If any irregularities are found in the books or papers for the 6 months immediately preceding, such inspector is then permitted to go back to the period of 6 months immediately preceding that, etc. If, however, everything is regular for the first 6 months preceding, that operates to prohibit inspection prior to that time. These inspectors get a considerable percentage of fines assessed and collected, and are consequently quite zealous, so that it is profitable to obey the stamp law scrupulously.

Necessity for Mexican Corporation Doing Work.—On taking charge of the affairs of the Mexican Co., the writer found that up to that time work done in Mexico had been paid for on the American side of the line through the C. D. Co., and in this way no Mexican stamps were required for payrolls, time checks, etc. In other words, the C. D. Co. had its forces go over into Mexico and do work on the canals of the Mexican Co. directly. As this was obviously contrary to the spirit of the Mexican laws on the subject, arrangements were made at once whereby the Mexican Co. did all work in Mexico and billed the C. D. Co. therefor at actual cost, the C. D. Co. turning over all materials and supplies required on the Mexican side of the Line at its expense.

Mr. A. F. Andrade, now Depositario for the Mexican Co., and Assistant General Manager of the Inter-California, was made General Agent of the Mexican Co., and was in charge of all negotiations between that corporation and the Mexican Government, and to his tact, energy, and ability is attributed the relatively small amount of irritation and delay encountered.

Occasionally, rules and regulations had to be disregarded, and this was done when it was deemed quite necessary, knowing that the local officers would report such infractions of the laws, but that the higher officials would view such infractions very sensibly when sooner or later brought to their notice with full explanations. For example, before permission was given to run trains into and out of Mexico after dark, a serious situation developed just at sundown, immediately requiring rock at the Lower Heading, and the Mexican officials at the Boundary Line would not permit trains to pass. Their protests were disregarded, for while the officials under the circumstances could not act otherwise, it would have been folly not to have disregarded their orders, considering the urgency of the matter. Proper explanations were at once made, and the company was not criticized in any way for the action.

Difficulties in doing work in Mexico are largely due to ignorance of Mexican conditions, customs laws, and personal characteristics, and doubtless are no greater than a Mexican would encounter in doing work in the United States. It is very desirable for the highest officer in charge of work to speak Spanish well, as minor Mexican officials are far more impressed with a statement coming from him than from any subordinate officer.

Methods of Diversion of River Through Rockwood Head-Gate.

The triangular space between the two faces of the **A**-frame and the horizontal cross-bracing of the wooden head-gate was made into a long pyramid, by flooring the bottom and sides, which was filled with sand taken in by wheel-barrows, in order to give additional weight to the gate in resisting the buoyant effect of the water.

By August 5th the discharge of the river had fallen to 24 500 sec-ft., and directly beside the Rockwood Head-gate the receding waters had exposed sand bars on each side of the main channel—the situation being as represented by Fig. 15. When these sand bars had

dried sufficiently, teams were used in throwing up an embankment on the line of the diversion dam. Brush jetties were also used to narrow the channel, the *Beta* assisting. In a little more than a week the stream was narrowed to 600 ft., the river gradually falling. Work was then begun on weaving a brush mattress, 100 ft. wide up and down stream, and sinking it on the bottom of the river. The decks of the barge, *Silas J. Lewis,* were cleared and skids were rigged thereon; ½-in. steel cables, 8 ft. apart, were anchored to "dead men" in the north bank and unwound from spools beneath the skids, such cables constituting the longitudinal strength of the mattress; and to these were fastened brush fascines averaging 18 in. in diameter and 100 ft. in length. These fascines were built up between vertical pins at the upper end of the skids and bound with baling wire, and as they were completed they were pushed down to the last one in the mattress and sewed to it and to the supporting cables with ⅜-in., 9-strand, galvanized-iron cable and cable clamps. Fig. 1, Plate CXV, shows the method of sewing and fastening. When a length of mattress equal to the width of the barge was completed, the barge was slowly pulled from under it, and it caught the silt and at once settled heavily to the bottom. No kind of weighting whatsoever was required. Another barge width of mattress was then woven and sunk, and so on. Figs. 1 and 2, Plate CXV, show the method of constructing the mattress and the number of men employed.

It required 20 working days, with two shifts, to weave and sink two mattresses, one on top of the other, across the bed of the stream, or a total of 1 300 ft.; thus the average rate was 65 lin. ft. or 6 500 sq. ft. daily. The work went ahead without interruption or difficulty except that once the anchor lines controlling the barge were not handled with sufficient care and the first layer of mattress was not sunk across in a straight line, but curved down stream in the middle perhaps 20 ft. at the maximum point. This, however, was not important.

The prevailing idea as to the necessity for such bottom protection in the river may be better realized from the fact that several engineers with the longest experience on the river joined in urging that a solid canvas back be sewed on the under side of the mattress. It was feared that the water might start a wash through a break in the mattress, that such a stream would carry the sand from below, cause a depression for the mattress to span, and result in breaking it when

weight should be put on above. This, however, was deemed unnecessary.

While the mattress work was being completed, a 4-pile railroad trestle with 10-ft. bents was started across the center line of this foundation, decked, and a railroad track built thereon. This trestle was driven from both ends, and was ready for the passage of trains on September 14th, 6 days after the completion of the mattress. In the mean time, the earthwork across the north sand bar had progressed sufficiently to connect the rails, so that trains could run out on the trestle. On the south side, the jetty work and the *Beta* had built up a sand bar on which a frame trestle on mud-sills was erected, connecting the earth embankment on the south sand bar to the trestle, thus affording tail room for trains. This frame trestle was filled in with material from the clay pit at Andrade.

At this stage, brush fascines were put in between the bents of the trestle over the channel, laid longitudinally with the stream, and sunk by rock from the quarry at Andrade. The rock was loaded into "battleships" with a steam shovel, hauled down, and dumped from the trestle. In this way a difference of 6 ft. in water elevation above and below this diversion dam was attained with no difficulty whatsoever.

Meanwhile the by-pass in which the Rockwood head-gate stood was being enlarged in several ways. The *Alpha* had cut a small channel from the crevasse to the gate from above and from below, through the solid ground, and the *Beta* had enlarged these cuts until it was taken over to assist in the jetty work on the south side of the river. A small ditch was cut with teams and scrapers across the sand bar, as an extension of the down-stream end of the by-pass. This channel was excavated to the water-table with Fresno scrapers, and made as narrow as possible, reliance being placed on enlarging it by the erosion of the water. In two or three places adobe deposits of considerable extent were found, and in these dynamite was used, as already explained.

The steamer, *Searchlight,* was anchored in the upper by-pass for two or three days with its rear end against the bank and the stern wheel kept going as fast as possible. This greatly hurried the erosion. The increasing head on the diversion dam aided these methods of enlarging the capacity of the by-pass until on October 10th only

FIG. 1.—WEAVING BRUSH MATTRESS.

FIG. 2.—DOWN-STREAM END OF BRUSH MATTRESS ABOVE WATER BECAUSE OF SILTING ACTION.

about 1 450 sec-ft. of the river's total discharge of 14 300 sec-ft. was not going through the gate.

The alignment of the by-pass was unfortunate, as it had quite a sharp curve, and the upper end left the river at a sharp angle. At this point cutting began, and to prevent it a small brush mattress was woven and weighted down with rock.

It was soon seen that, with the 4-ft. openings between them, the A-frames of the gate caught the drift in the water very badly. Anticipating this, cables had been stretched across the entrance of the by-pass and fitted with grab-hooks, like fish hooks on a trout line. These grab-hooks were of $\frac{3}{8}$-in. wrought iron fastened with from 6- to 8-ft. lengths of sewing cable to the cable spans at intervals of about 8 ft. It was hoped that these would catch drift where it could easily be removed, and prevent trouble at the gate. However, they did very little good.

When the current through the gate increased to 6 or 8 ft. per sec., a scour developed both above and below. Soundings showed that the scour below the gate was not at all serious, but was really far less than had been anticipated. The eddies at the ends of the gate caused side-cutting, as is always the case, but really nothing alarming. The scour above the gate, however, was surprisingly great; some was expected, but not nearly as much as occurred. Brush and rock extension of the apron, as shown on the plans, had not been put in as it had been the intention to use rock from Andrade in lieu thereof. When soundings, which were taken frequently, showed that the by-pass bed was eroded to the level of the floor of the gate, approximately 1 000 cu. yd. of rock were loaded on a barge which was swung in front of the gate and held by cables until unloaded.

Failure of Wooden Head-Gate.

On October 3d a serious settlement of the earth filling in the north abutment suddenly occurred. Excavation was at once made to ascertain the cause, and some small leaks in the end wall on the up-stream side of the A-frame were found. These were stopped up, and the earth was leveled to only a few feet above the water surface on the outside. Two days later the lower wing-wall in this same abutment spread out at the bottom on the west side, as shown in Fig. 1, Plate CXXII. The gate itself buckled up about 0.3 ft., about one-

third of its length from the abutment, such buckling apparently occurring very slowly within 24 hours, ending on October 5th. These signs of weakness were accompanied by the tearing up of the upstream apron in relatively small sections, which were at once thrown against the A-frames by the current. With great difficulty these were taken out piecemeal, and then only in part. These, together with the drift which accumulated, caused a head of 4.4 ft. on the gate on October 11th. At this time the discharge through the gate was about 12 000 sec-ft.; the maximum discharge through it was about 13 000 sec-ft. on October 8th.

These indications of weakness showed that it would not be safe to use the gate after closing the break, and that it would be very fortunate if it held until this could be accomplished. Furthermore, the drift made it very difficult, if not impossible, to set the flash-boards. Accordingly, on October 5th, a pile bridge was begun just above the gate and connected with the track to the south by a frame bent trestle supported on mud-sills—the same construction as had been utilized on the south side of the channel. This trestle was finished in the morning of October 11th, and it was intended to dump rock from it and fill up the gate in this way and not attempt to use the flash-boards.

When the first rock train was slowly pushed over the trestle, at 11 A. M., three bents of the frame trestle settled and wrecked the train, fortunately injuring no one seriously. Just why construction which on apparently worst ground on the south side of the main channel was entirely satisfactory should have failed here, is not known—things happened thereafter too rapidly to find out. At any rate, had the trestle stood and had the large number of loaded "battleships" held ready been dumped, the writer has always believed that the head-gate would not have failed utterly. Be that as it may, at 2: 30 P. M., without any warning, the gate suddenly buckled up at a point about one-third of the way from the south abutment, and the larger portion—from there to the other abutment—floated down stream about 200 ft., where it lodged. The remainder of the gate stayed in place, although it settled in the central end. When the gate went out, the 4.4-ft. head above it caused a destructive wave of water, carrying large quantities of drift and débris from the wrecked gate against the railroad trestle crossing the by-pass about 300 ft. below. In about 5 min. this

FIG. 1.—GRADE RECESSION IN NEW RIVER NEAR CALEXICO. MAXIMUM RATE OF RECESSION, 1 FOOT PER MINUTE. DROP, 28 FEET. JUNE, 1906.

FIG. 2.—THE HIND DAM, PASSING 7 000 SECOND-FEET. HEAD, 10 FEET.

damaged the trestle seriously, and would have marooned a locomotive and train standing on the south side of the by-pass had not the engineer taken chances and pulled across before the piling began to go out.

The pond above the diversion dam extended some distance up stream and contained a large quantity of water which had to run out before the flow through the by-pass was reduced to the discharge of the river. By the time this occurred, considerable inroad had been made at the point where the upper by-pass left the river—which had been protected by a small brush mattress—and for a time it threatened to work down to and through the earth portion of the dam. Aggressive work was centered there, and such action was finally arrested.

Closing the Break with Rock Fill Barrier Dams in Series.

The lowering of the water above the diversion dam left it dry, except for a surprisingly small quantity of leakage, and enabled examination of the rock fill which had been produced by an ever-increasing proportion of rock with respect to brush. This condition of affairs seemed to indicate that the reasons urged why a rock fill dam of considerable height could not be built in a running stream were not altogether strong, and suggested the possibility of very quickly controlling the situation with a series of rock fill dams, each of which should sustain a head of not more than 4 ft. This particular dam had stood successfully a head of 6 ft. without any of the troubles prophesied for constructing rock fill dams in streams. Furthermore, the tracks of the Southern Pacific Company on the Salton Sea were in an extremely critical condition, and the southern transcontinental line would soon be interrupted, at an estimated cost of $1 000 000 a month. It was obvious that, if this were to be prevented, very quick action was necessary, and if hope should be abandoned, withdrawal of financial support in controlling the river was almost a certainty. Furthermore, other plans of controlling the situation possessed most serious difficulties, as already explained.

As a matter of insurance, however, a rush order was wired for additional sewing cable for building a diversion dam across the Colorado directly opposite the concrete head-gate, exactly as had been done successfully opposite the wooden head-gate to divert the river through the former structure, and as had been done with the other,

trusting to dynamiting, dredging, erosion, etc., for enlarging the 4 miles of Main Canal thence to the break. This done, the trestles across the by-pass, above and below where the wooden head-gate had been, were repaired, and a third trestle, 30 ft. above the lower one, was hurriedly thrown across this stream, which was carrying the entire flow of the river, the waterway through the opening of the gate being only 120 ft. wide.

Such method of closing the break and forcing the river down the old channel by three rock fill barrier dams in series was therefore considered problematical only because there was no mattress under any of them, and the brush mattress idea had always been regarded as essential. The branch railroad from Hanlon's Junction to the Lower Heading was now in excellent condition, and the Andrade quarry was sufficiently developed to permit the use of the two steam shovels, producing about 5 000 cu. yd. of rock daily, by working night and day. It was felt that with these facilities, together with the rock which could be obtained from quarries within a distance of 400 miles to the east and west, rock could be put into the stream faster than the water could carry it away.

As a matter of fact, these three dams were built up so rapidly and successfully that only 10% of the water was going through the by-pass by October 29th, most of the remainder—8 600 sec-ft.—going over the diversion dam with the mattress foundation. Here, a secondary trestle, with 4-pile bents, 15 ft. apart, parallel to and 30 ft. up stream from the first, had been rushed, and from the two a rock fill dam was completed, turning all the water—9 270 sec-ft.—down the old channel and actually closing the break on November 4th. That is, after working on other lines continuously for 15 months, the stream was controlled by a rock fill dam in 24 days. In other words, the rock fill barrier dam plan, which had not been advocated, or indeed seriously considered, by a single man, proved to be a very simple and efficient, though expensive, method of re-diverting the river. The fact that there was a very substantial brush mattress foundation, however, was deemed by many as of vital importance.

Leakage through the structure was stopped by dumping "battleship" trainloads of gravel from the Mammoth gravel pit and clay from the clay pit, the whole being puddled with fire streams. The *Beta,* which was kept above the diversion dam in order to be taken up

FIG. 1.—SEALING THE HIND DAM WITH GRAVEL AND CLAY BY HYDRAULIC JETS, NOVEMBER, 1906. NOTE SLOPE OF DOWN-STREAM SIDE OF ROCK FILL DAM.

FIG. 2.—GENERAL VIEW OF THE SECOND BREAK, JANUARY 20TH, 1907.

the river and used in the intake above the concrete gate, was used in widening the up-stream toe of the dams.

A week and a half after the failure of the wooden head-gate, the success of the series rock fill dam plan seemed assured. The *Alpha* had finished its trip up the Main Canal and cut into the excavation in which the concrete head-gate had been built. The intake from the river to the concrete head-gate was completed, and by October 29th the river at this point had been raised approximately 4 ft. by operations at the break. The dam holding out the river here, and those which had been left by the *Alpha* on its way up, were blown out, and water commenced to flow through the concrete head-gate and Main Canal into the Alamo channel below the diversion operations. The initial discharge was about 150 sec-ft., and had increased but little when the river re-diversion was complete. At that time (November 4th) the water height at various points was as follows (C. D. Co. datum):

Above the dam................	113.0	ft.
Below the dam................	97.3	"
Opposite concrete head-gate....	114.5	"
Floor of concrete gate.........	98.0	"

By November 15th only 300 sec-ft. were flowing in the Main Canal, the fall of 17 ft. in these 4 miles not having resulted in much erosion, because of several stretches of adobe deposits, though the current was quite strong. Dynamite was used liberally, and by December 5th the grade recession was within 1 mile of the head-gate. In this way continuity of supply into the valley was kept up, and the water users suffered relatively little inconvenience.

In making the first closing, rock was unloaded from the three trestles across the by-pass and two trestles over the main channel. Records were kept daily of car loads of rock from Andrade and from the distant quarries unloaded from each trestle, but this record, unfortunately, has been misplaced, and the totals obviously signify nothing. As the quantities of various materials used during the entire period from August 1st to November 4th may be of interest, they are given in Table 13.

Completing the Hind Dam.

The dams across the break and the by-pass were hurried to completion with material from the Mammoth gravel pit and the clay

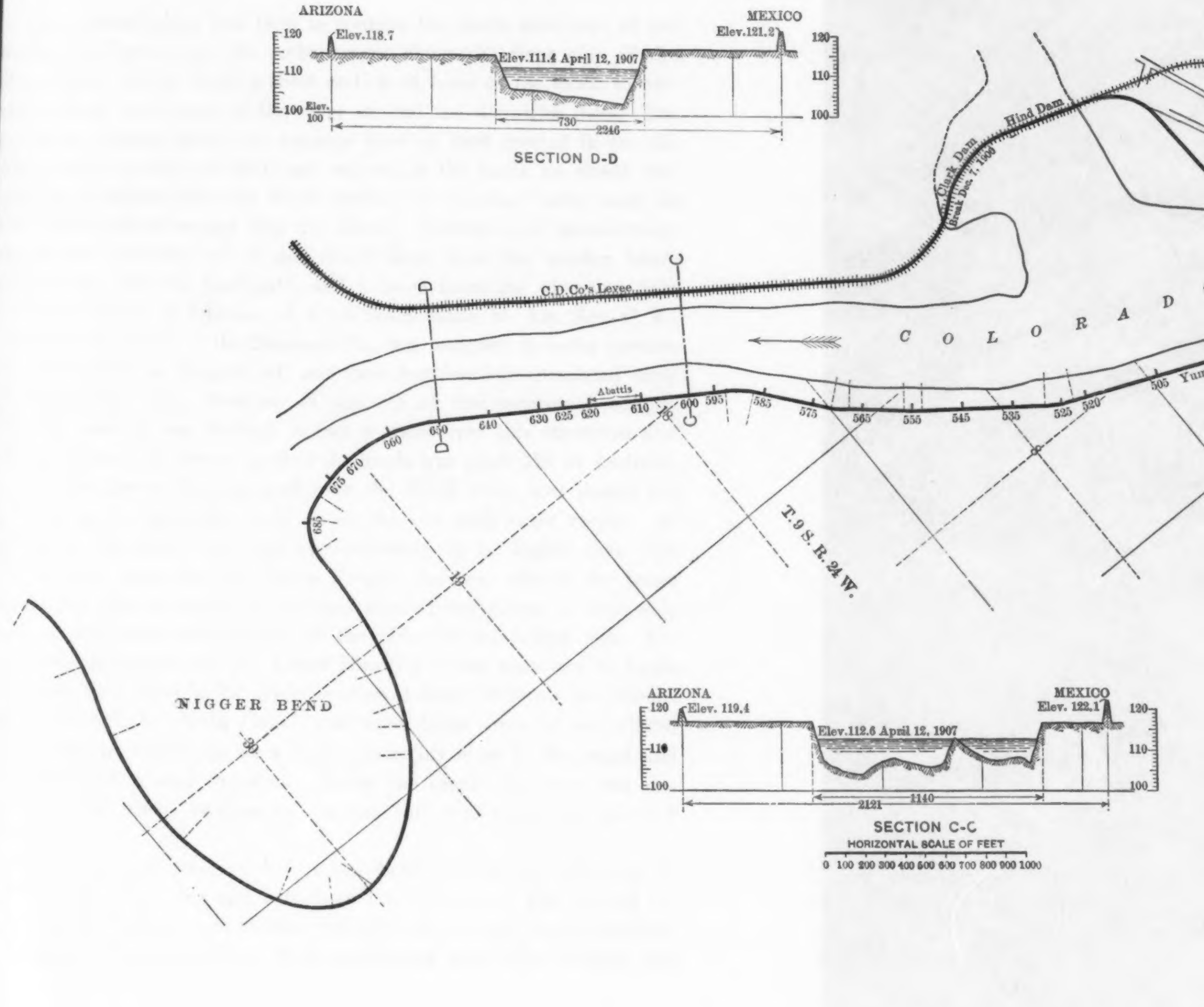
pit at Andrade. It was decided that the structure should have a top elevation of 124, and that meant increasing its height fully 8 ft. The tracks over the trestles were raised so rapidly that no attempt was made to recover the stringers or caps.

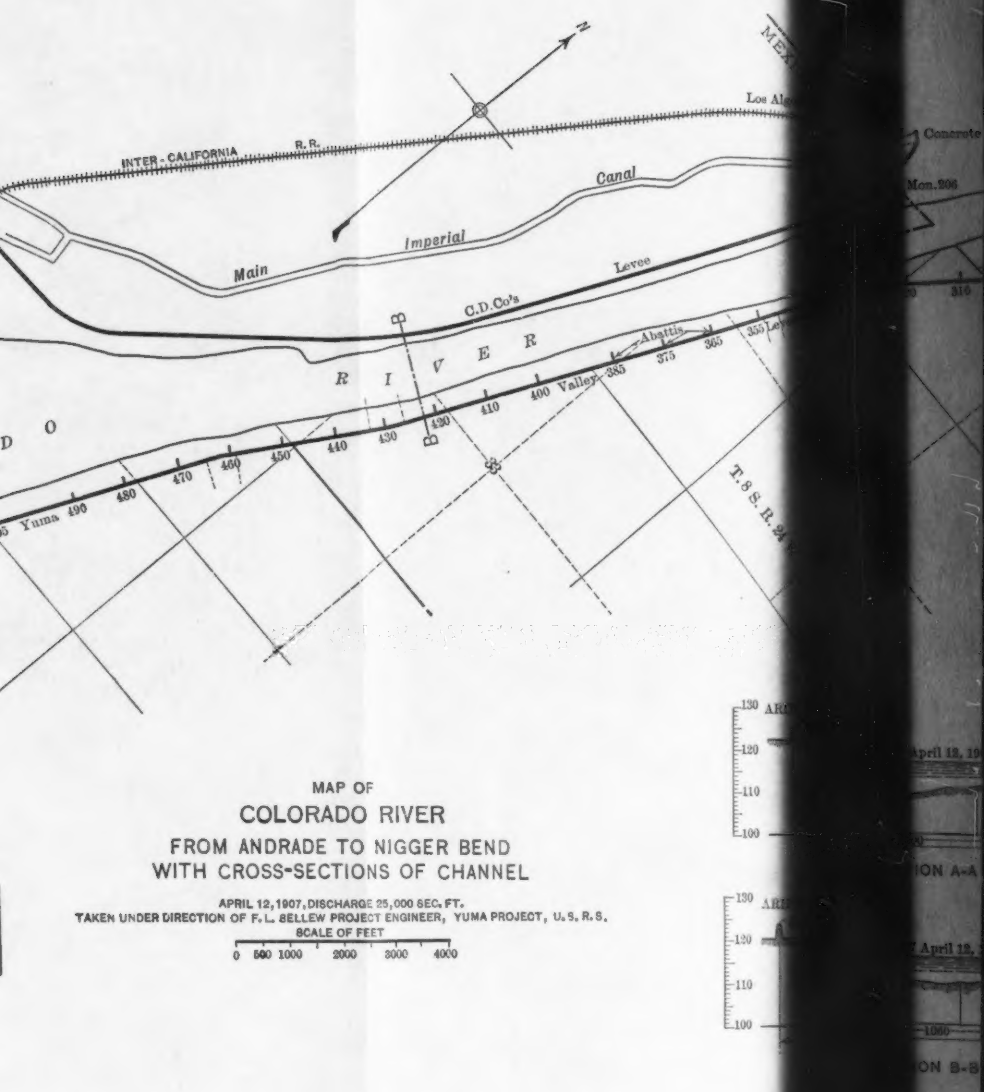
TABLE 13.—APPROXIMATE DATA OF CONSTRUCTING DIVERSION WORK ON COLORADO RIVER.

2 200 cords of brush and 40 miles of steel cable used in mattresses and shore protection.
3 800 ft. of railway trestle.
15 200 ft. of 8 by 17-in. Oregon pine stringers.
1 100 piles.
1 690 cars (50 000 cu. yd.) of rock (90% used from October 11th to November 4th).
841 cars (32 000 cu. yd.) of gravel.
808 cars (33 000 cu. yd.) of clay.
200 000 cu. yd. of earth, placed by teams.
200 000 cu. yd. of earth, placed by dredges.
200 to 500 head of mules and horses working from July to November 20th.
200 men in June, increasing to 1 000 men on November 4th.
Discharge of river, June 27th, 99 200 sec-ft.
Discharge of river when actual work of constructing channel was begun, August 6th, 24 400 sec-ft.
Discharge of river on November 4th, when final closing was made, 9 275 sec-ft.
Elevation of water above dam, 113.1 ft. above sea level (C. D. Co. datum).
Elevation of water surface below dam, 97.30 ft above sea level (C. D. Co. datum).
Total head on closing, 15.8 ft.
Elevation of water surface above dam one week after closing, 112.60 ft. above sea level.
Elevation of water surface below dam one week after closing, 95.85 ft. above sea level.
Total head on dam, November 11th, 16.75 ft.

The tracks were gradually pulled together to a final 13 ft. between center lines, which helped somewhat, but the proper side slopes were chiefly obtained with fire streams, five 1¼-in. nozzles, each throwing about 225 gal. per min., being used. The mixed materials as dumped assumed a slope of about 1¼ to 1, as a rough average, and these were very quickly and cheaply flattened down hydraulically to about 2¾ to 1 on the river side and 2 to 1 on the land side. Furthermore, the slopes were really well finished with very slight additional care and expense.

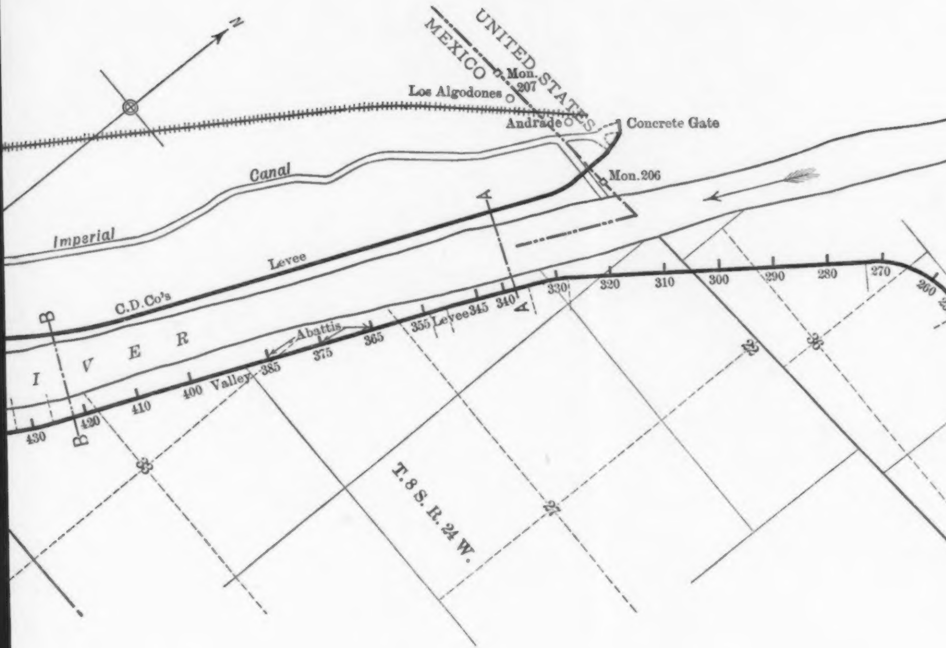
In its final form the dam has about 400 ft. of 15° curve at the north end, and 2 275 ft. of tangent; the dam is connected at each end with the levees extending along the river. At the north end there are 200 ft. of high dam with a rock fill core to within 8 ft. of the top —where it crossed the by-pass. A little more than half way toward the other end there is 600 ft. of another high stretch with a rock core on brush mattress foundation; the remainder is from 16 to 20 ft. high. This is known as the Hind Dam, so called after Mr. T. J. Hind, Superintendent of the work at the Lower Heading after June 1st, 1906, to distinguish it from the Clarke Dam, closing the second


ARIZONA
MEXICO
Elev.118.7
Elev.121.2
Elev.111.4 April 12, 1907
730
2246
SECTION D-D
Hind Dam
Clark Dam
Break Dec. 7, 1906
C.D.Co's Levee
C O L O R A D
Abattis
T.9 S. R. 24 W.
NIGGER BEND
ARIZONA
Elev. 119.4
MEXICO
Elev. 122.1
Elev.112.6 April 12, 1907
2121
1140
SECTION C-C
HORIZONTAL SCALE OF FEET
0 100 200 300 400 500 600 700 800 900 1000



MAP OF
COLORADO RIVER
FROM ANDRADE TO NIGGER BEND
WITH CROSS-SECTIONS OF CHANNEL

APRIL 12, 1907, DISCHARGE 25,000 SEC. FT.
TAKEN UNDER DIRECTION OF F. L. SELLEW PROJECT ENGINEER, YUMA PROJECT, U. S. R. S.
SCALE OF FEET
0 500 1000 2000 3000 4000



F

RIVER

NIGGER BEND

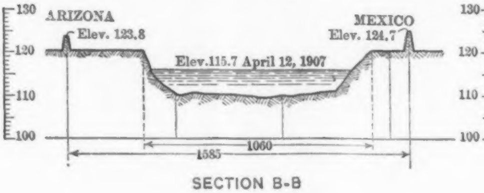
NS OF CHANNEL

E 25,000 SEC. FT.

CT ENGINEER, YUMA PROJECT, U. S. R. S.

EET

3000 4000



SECTION A-A

SECTION B-B


270
260
251
Yuma
235
230
225
221
Valley
31
T. 16 S. R. 22 E.
170
Levee
32
Abattis
95
33

break, so called after Mr. C. K. Clarke, Superintendent of the second closing, December 20th, 1906, to February 20th, 1907. About 80% of this dam was complete on December 7th, 1906.

Levee Construction.

The original plans had been to connect the north abutment of the wooden head-gate with the embankment along the river side of the Main Canal, and to build a short section of levee to the south to prevent a flank movement of the river around the diverting dam. The enormous channel which the summer flood of 1906 created in the old Alamo made it obvious that, not only must the break be closed, but that, by a rather elaborate levee system, all overflow water must be kept from getting around into the Alamo. Surveys and examinations showed the necessity of an additional levee from the wooden head-gate to the concrete head-gate, and a levee from the diversion dam south for from 5 to 6 miles. J. C. Allison, Assoc. M. Am. Soc. C. E., Assistant Engineer of the Mexican Co., was assigned to make surveys for these levees on August 1st, and their location was completed early in September. The elevation of the top of the concrete head-gate was 124, and it was decided to put a track over this structure and extend it down the levees, so that the grade was made 126 at Andrade, 124 at the Lower Heading and over the Hind Dam, and thence for 4 miles south, generally 6 ft. above the old high-water marks. At all points the grade was kept approximately 2½ ft. higher than that of the levee opposite the Yuma Project, because, should the latter break, the damage would be far less than if the levees on west side were to fail, with re-diversion of the river to the Salton Sea. Between the head-gate and the Lower Heading it was necessary to locate the levee very close to the river, because it must obviously be between the river and the Main Canal, and some large areas of bad adobe, damp, and impossible to work with teams, lay close to the canal and extended well toward the river. Below the break, the levee was also close to the river, because of similar soil conditions for about 4 miles.

The levee was designed with a top width of 8 ft. and slopes of 2½ to 1 on the river side and 2 to 1 on the land side. The ground for the base of the levee was cleared and grubbed, but no "muck-ditching" was done. The desirability of muck-ditching was fully realized, and

it was a part of the levee design. Experience in the valley had always shown that, not only ditch and canal banks, but low borders of irrigated fields, etc., leaked badly when water was first applied. Indeed, interesting cases were cited of water in considerable volume disappearing into the ground for several days, doubtless flowing away under the surface through partly opened cracks of buried layers of cracked adobe.

On the other hand, the money supplied by the Southern Pacific Company was for closing the break, and only for that purpose, until the re-diversion of the river was assured. No narrow construction was placed on this, to prevent building levees at all, but it was not considered proper to incur any avoidable expense in this direction until it should have been clearly demonstrated that it was physically

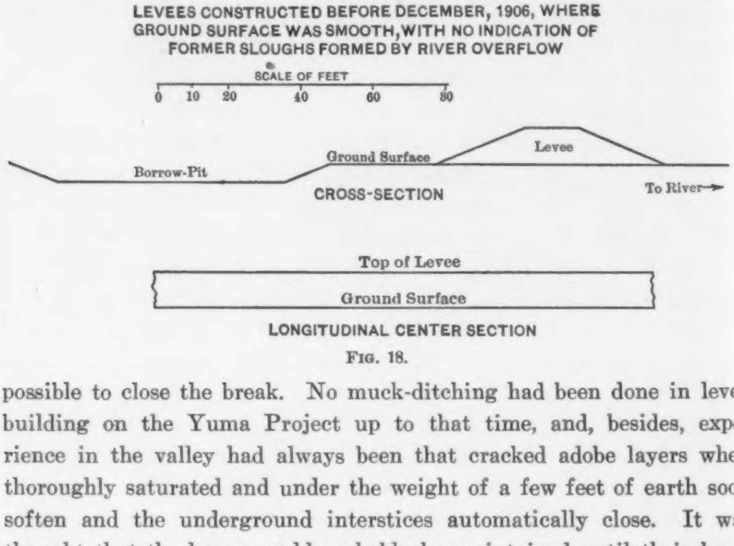


FIG. 18.

possible to close the break. No muck-ditching had been done in levee building on the Yuma Project up to that time, and, besides, experience in the valley had always been that cracked adobe layers when thoroughly saturated and under the weight of a few feet of earth soon soften and the underground interstices automatically close. It was thought that the levees could probably be maintained until their bases would thus soak up tight, although it was certain that they would leak like sieves when water first came against them. Thus it was ordered that muck-ditch work be omitted.

Material for the levees was taken from borrow-pits on the land side. It was fully realized that this was not in accordance with the usual practice, but it was decided on after careful consideration of the advantages and disadvantages. The location of the levee

was forced very close to the river for a great portion of the way, and the levees of the Yuma Project on the opposite side were also so close to the stream that the distance between was in many places only 1 400 ft.—an exceptionally narrow waterway for such an unruly stream as the Colorado. As it was certain that the current at flood stages would be very great in such sections, it was extremely desirable not to disturb in any way the rank vegetation between the river and the levee, as it could not help but greatly break up and retard currents and thus protect the levee from erosion.

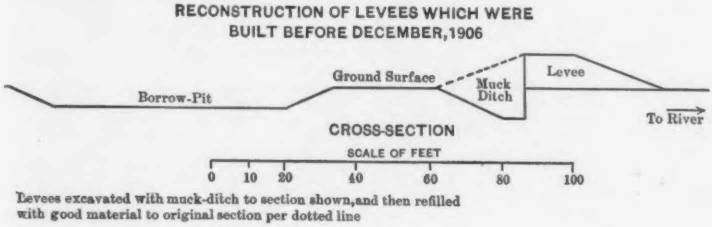


Fig. 19.

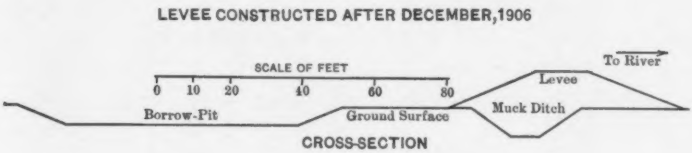


Fig. 20.

Experience with the levees of the Yuma Project showed that the hope that borrow-pits would be silted up was vain, and instead, that they would be cut together to form a continuous canal having eddying currents below the traverses during high floods, unless extensive brush abatis work was used. This sort of protection was deemed very unsatisfactory, because, though the Mexican Co. actually owns the land on which the levees were located in Mexico, it is practically impossible to exercise very much control over the Indians, owing to the indifference of local Mexican authorities. The Indians have always utilized any overflowed areas along the river as they wished, for their little garden patches, and these levees must absolutely cut off such water. For a long time it was utterly impossible to keep these nomads from planting seed in the borrow-pits, where the ground remains

LEVEE SECTION
RECOMMENDED BY CONSULTING ENGINEER BOARD
OF RECLAMATION SERVICE

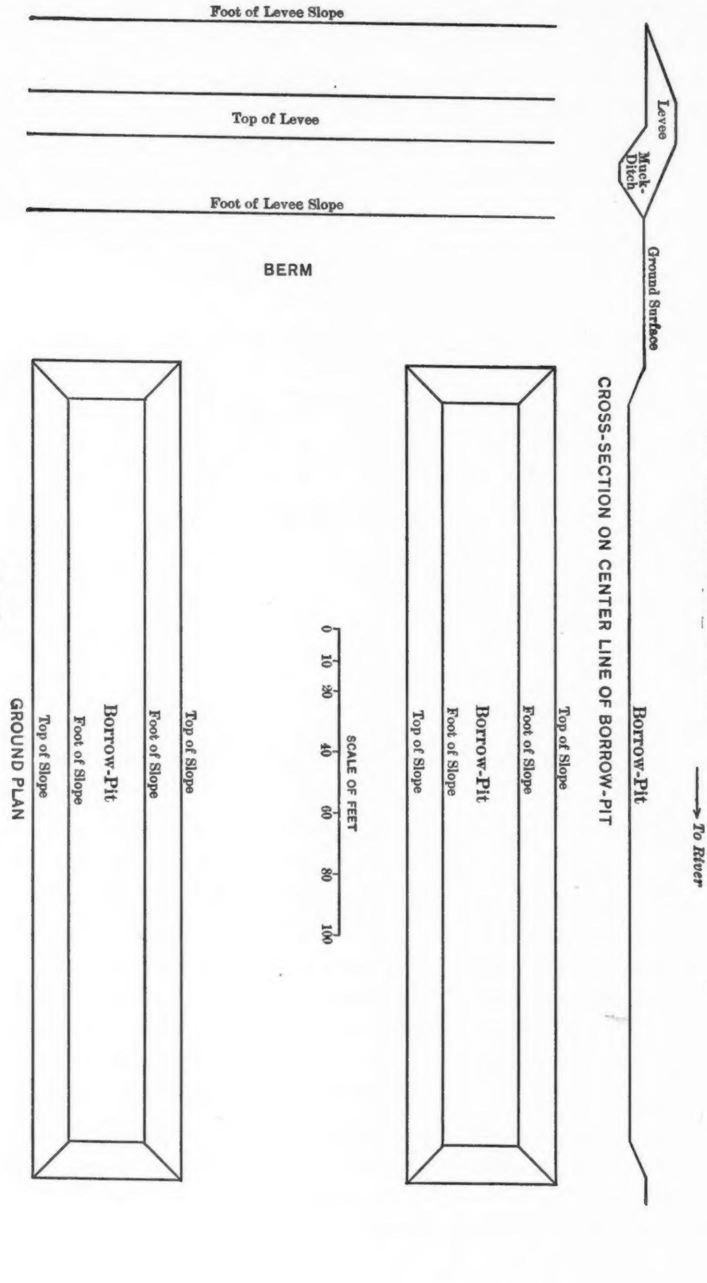


FIG. 21.

wet the longest, and from destroying all brush growths that start therein. It was considered impracticable to maintain brush abatis work, which, when dry, would only make it easier to burn off the area in front of the levee for a garden clearing.

On the other hand, the land spoiled in making borrow-pits was of little value, being non-irrigable under existing conditions. There are no quicksand pockets above the water-table in that region, and, the soil being alluvial silt with more or less sand intermixed, there was consequently no fear of water-soaked material running, such as occasionally causes levee trouble elsewhere. There is also along the river no surface soil crust, which it is undesirable to disturb in levee building.

The only pertinent objection to land-side borrow-pits in this case, therefore, seemed to be the matter of increased total head, which it was decided did not outweigh the advantages of an undisturbed rank vegetation as a protection against the erosion of the water slope by swift currents.

These levees were built by the Shattuck and Desmond grading outfit on force account, with the intention of changing to a yardage basis as soon as possible. On December 6th, 1906, about 1½ miles above and below the dam, respectively, had been completed, 5 miles more were under construction, and the ground was cleared for another 2 miles.

Situation of the California Development Company and the Mexican Company.

About November 15th, 1906, the various operations along the river were making satisfactory progress, and the writer for the first time since June 1st left the river, hurriedly investigated the condition of the C. D. Co. and the Mexican Co. and reported his findings toward the end of that month. As a result of this, when the second break occurred a few days later, further advances by the Southern Pacific Company were advised against, and President Randolph concurred and so reported to Mr. Harriman. The fact that such a decision was made by the Harriman interests unqualifiedly has not been generally accepted without some mental reservations. It may be interesting, therefore, to make some excerpts from this report. The balance sheet for the combined C. D. Co. and Mexican Co. was approximately as follows:

ASSETS.

Real estate (chiefly in Mexico)		$545 037.26
Stocks (chiefly unsold water stock)		175 600.00
Plant:		
Machinery and equipment	$179 621.82	
Branch railroad track	63 000.00	
Canals in Mexico	375 000.00	
Canals in United States	308 616.37	
		926 238.19
Accounts receivable (chiefly notes secured by water stock).		235 137.02
Total		$1 882 012.47

LIABILITIES.

S. P. Co.—Audited bills and interest		$1 532 595.73
General audited bills and interest		73 786.72
Bonds and accrued interest		515 200.00
		$2 121 582.45
Damage claims (probable):		
New Liverpool Salt Co	$50 000.00	
Land owners	200 000.00	
Water Companies Nos. 6 and 8	500 000.00	
S. P. Co	1 000 000.00	
Inter-California R. R.—S. P	250 000.00	
		2 000 000.00
Total		$4 121 582.45
Net liabilities		2 239 569.98

The possibility of extending the canal system and selling additional water rights was discussed, and the opinion was offered that such possible returns would probably just about suffice for building a canal from the river into the valley, to take the place of the Alamo channel, which might be necessary in 5 or perhaps not for 20 years; and for building new controlling works on it in the valley.

The conclusions were that maintenance and operation expenses, properly estimated, would take up the returns from the sale of 600 000 acre-ft. per annum. The cost of protecting the region from the Colorado flood-waters was set down as problematic, probably averaging

$100 000 per year, and fluctuating enormously, and it was advised that the task of controlling the river was too serious a tax on the enterprise under any possible circumstances, while international complications would probably mean considerable delay in arranging for such work equitably and satisfactorily.

The Second Break.

On December 5th, 1906, a severe flood came down from the Gila, as shown by Plate CIX. Superintendent Hind and the writer, who were in Imperial Valley at the time, received telegraphic notice from the water-shed, and went at once to the river. For reasons already explained, trouble was expected from water getting under the levees, because no muck-ditch protection had been provided, and a large force of men was detailed to watch the sections for a mile on either side of the Hind Dam day and night as soon as water came even near the river toe of the levee. Information from the upper stations throughout the Gila water-shed frequently indicate floods which never materialize, and this was another case of the "truth itself" being not believed. There was so much accumulated work in the valley that there was no time to watch a discharge of 30 000 sec-ft., although a river stage which would test out and soak up the levees was obviously of great importance. Mr. Hind and the writer were in Yuma, returning to the valley, when the flood reached the Lower Heading, where the river began rising at midnight and rose at the rate of 1 ft. per hour until the peak was reached early in the morning. At 3:30 A. M. Mr. Hind and the writer left Yuma on the work train, reaching the Lower Heading about 5:15 A. M. and found three serious and distinct breaks within 100 yd., the first one being about 2 400 ft. from the south end of Hind Dam. In addition water was finding its way under the levee in about ninety other places, within the stretch where the water reached the toe of the levee, or about ¾ mile above and an equal distance below the dam. Mr. J. Calvert, General Foreman, had fully obeyed instructions, and when the water began to reach the toe of the levees at the lowest point, had commenced work with his force of about 75 men, doing all that seemed possible. The trouble was not that any one break could not have been easily handled, but that so many points of weakness were developed practically at the same time. Indeed, it is really remarkable that the situation got beyond control in only these three places.

A part of the general arrangements with the Inter-California Railroad was its use of the Hind Dam. On the south end of this dam the proposed alignment turned a small angle to the right, and it was planned to have the fill for the next 2 miles without any openings—constituting thus a spur levee to prevent any water which passed through the main levees from reaching the old channel of the break beyond the dam. Had this fill been ready, or had the first serious break been 500 ft. farther along, sand bag diking across the traverses and out into the brush to force the water to spread out and follow an old and well-defined swale which entered the Alamo 2 miles beyond, would have been easily possible. In either case, the damage would have been limited to losing less than 1 000 cu. yd. of levee section.

As it was, however, the water coming through the breaks filled the borrow-pits on the land side, overflowed the intervening traverses, and, as the land in general sloped westward, over-topped the last traverse by the channel of the break below the dam, and caused a rapid grade recession from there, following the borrow-pits through the nearest break. When Mr. Hind and the writer reached the scene, the first of the three breaks was beyond control, and the situation was hopeless.

By the time the grade recession had reached and passed through the break, the flood had crested, and the water had risen against the levee to a depth of about 4 ft. The water rushing through rapidly increased, and cut into the far side of the bank, and was deflected and began cutting into the land side of the levee. This soon breached the dike about 1 000 ft. from the end of the Hind Dam. This breach became the main break, and was rapidly widened and deepened until, within 24 hours, the old channel was again entirely dry and the river had been re-diverted into the Salton Sea.

The men of the grading outfit engaged on levee extension work 3 miles down the river were flooded out, and the steamer *Searchlight* was sent to relieve them. The re-diversion into the Salton Sink occurred so rapidly that the steamer was left grounded there in the old channel, and inasmuch as it was the only craft on the river not controlled by the contractors on the Laguna Weir, this was a serious matter.

The Southern Pacific Quits.

This disaster brought to the higher authorities of the Harriman Lines a thorough realization of the size of the great task of controlling

FIG. 1.—LEVEE FAILURES RESULTING IN SECOND BREAK, DECEMBER 7TH, 1906. NOTE THE SHORT LENGTH OF THE TWO GAPS.

FIG. 2.—EFFECT OF FLOOD OF DECEMBER 7TH, 1906, ON LEVEE OF U. S. RECLAMATION SERVICE, OPPOSITE LOWER HEADING.

the Colorado. The imperative need of invulnerable levee construction for at least 10 miles along the river was made evident, and the difficulty and cost of building and maintaining such a system was emphasized. Entirely aside from the very great cost of bank protection work to prevent the breaching of levees through the side cutting of the banks, was the difficulty of building in such bad soil a line of defence which would be absolutely dependable.

The financial condition of the C. D. Co. and the Mexican Co., as just explained, was very bad, and, under the most favorable circumstances possible, the chances of the Harriman interests ever being able to get back very much of the moneys already advanced were extremely remote. In addition, however, it was apparent that an unusually efficient and expensive levee system would be required, the first and maintenance cost of which was too large a burden to undertake.

The stockholders of the irrigation properties notified the Southern Pacific management controlling them that the properties could not be expected to do such overflow protection work, and indeed should not pay more than a proportional part based on the total value of the property interests in jeopardy, especially in view of the immense amount of work which had already been done at its expense. Urging that the irrigation company had caused the menace (which may or may not be entirely the case) had not the slightest significance to the Southern Pacific interests, which were really the only ones with any funds, collateral, or equipment, and were in no possible sense responsible for any changes in physical conditions along the Colorado River, except to make them very much better than they otherwise would have been.

On the other hand, it was recognized that something would have to be done very quickly because the summer flood of 1907 would in all probability cause such grade recessions as to force a hurried exodus from Imperial Valley which would be without a parallel in history. The chances of such grade recession extending far enough to render the control of the river after that flood very much more difficult, were remote. The matter was made complex by the fact that all work had to be done in Mexico and practically all property interests in jeopardy were in America; and there were no provisions, State or National, to handle such a curious situation. Unless the river was turned and kept going

to the Gulf, the Southern Pacific would suffer the loss of its traffic from the Imperial Valley and would have to change its line and build 100 miles of track, but, to obviate these losses would certainly not justify it in undertaking to control the river single-handed.

Accordingly, the people of Imperial Valley were notified that, while the Southern Pacific would be very glad to place such equipment and organization as it had along the river at the disposal of any party who wanted to proceed with the work, and would be willing to contribute toward the expenses thereof in proportion to the value of its interests as compared to all others in jeopardy, it would not advance additional funds without a definite arrangement for being reimbursed. Work on a roadbed following —100-ft. contour was ordered and rushed to completion. The cost of grading was very small, and such a line would preclude the possibility of the interruption of transcontinental traffic by the Salton Sea for at least 4 or 5 years, during which time a line lying entirely above sea level could be economically constructed.

On December 13th a mass meeting of the people of Imperial Valley was held in Imperial, and subscriptions for river control work, totaling \$950 000 were made by various interests. These were the Imperial Valley Improvement Company (the practical successor of the Imperial Land Company) \$100 000; the Holton Power Company, \$100 000; the C. M. Co., \$250 000, and the directors of the mutual water companies together a bond issue of \$500 000. All these were made promising payment 90 days after the break should have been closed successfully, the railroad to assume all risk of the work.

While considering these subscriptions, it was urged in opposition that the mutual water companies might not be able legally to issue bonds or expend money for river protection work at all, or indeed that the people of the valley could raise money, except by individual subscription, for work to be done in Mexico. Requests were sent out in all directions, resulting in numerous civic and political bodies and authorities of the State wiring to President Roosevelt asking to have the United States Government act in the emergency. The President acted promptly, and as the result of telegraphic correspondence with Mr. Harriman, instructions to start work on the river were received on December 20th.

In the meantime the organization at Andrade and at the Lower Heading had been kept intact. The quarry was developed, sidings just across the border in Mexico were lengthened to 7 000 ft., and material and equipment of all possible kinds which might be needed were gathered in readiness to proceed whenever orders might be received.

Senator Flint's Bill.

Immediately after the holidays, Senator Frank C. Flint, of California, introduced a bill in the Senate providing for the appropriation of $2 000 000 to handle the situation. Under the provisions of this bill, whatever sum might be found just should be paid to the Southern Pacific Company for work then under way, and the remainder should be utilized to establish an irrigation project for Imperial Valley by the U. S. Reclamation Service. The idea was that the irrigation of American land in the Salton Basin and the regulation of the Colorado River were inseparably connected, and that as soon as the situation should be under control by the Southern Pacific Company, the entire matter should be turned over to the Reclamation Service for future handling. President Roosevelt, on January 12th, 1907, sent a special message to Congress severely criticising the promotors of the C. D. Co. and the management of the properties, and urging the passage of the Flint bill in order to relieve the settlers of Imperial Valley from the "injustice" they were enduring.

When the bill reached the House, Hon. S. C. Smith, Representative from the Eighth California Congressional District—in which Imperial Valley is located—opposed it, advising that he did so because of requests from his constituents in the valley. There can be no doubt that, with very few exceptions, the farmers in the valley objected to the bill, preferring the existing irrigation arrangements to those which would follow under the Reclamation Service,* and desiring governmental assistance in river protection work, and in that only. Largely due to Mr. Smith's efforts, the bill failed to pass.

Change in Organization.

In spite of the opinions of visiting engineers, experience during the first closing left little doubt that there would be any particular difficulty in making the second closing without any brush mattress

* Among the objections, the two most important were the probable increase in cost of water and the necessity for reducing individual holdings to probably 40 acres or less.

foundation. Even had such been deemed desirable and worth the delay in time, very little brush was available nearby because of the large quantity used for the first closing. It was felt that the work was standardized and consisted in throwing ordinary railroad trestles across the break and from them making rock fill dams in series. The levee construction work to be done, however, was very much of a problem in every way. Superintendent Hind was transferred to the levee reconstruction and extension, and Mr. Clarke, formerly Resident Engineer of the Tucson Division, Southern Pacific Company, came to the work as Superintendent of the second closing on December 20th, 1906.

At the same time, an entire change in the accounting system was ordered, effective December 7th, the date of the break. Prior to that time the work had been done by the Mexican Co., with material and funds supplied by the C. D. Co., and the latter corporation from time to time borrowed money from the Southern Pacific Company. This was changed so that the Mexican Co. was furnished money by Epes Randolph, Agent of the Southern Pacific Company, and the C. D. Co. had nothing to do with the matter whatsoever. On the American side of the line, the operations were exclusively under the name of "Epes Randolph, Agent, S. P. Co." The railroad furnished supplies and material to him under the same arrangements and conditions as it had furnished them to the C. D. Co., namely, at a cost plus 10 per cent. One marked difference, however, occurred in making these charges, namely, that after January 1st all freight bills were rendered at traffic rates instead of at ½ cent per ton-mile, this being made necessary by the provision of the Interstate Commerce Commission which took effect at that time.

Closing the Second Break.

The work of closing the second break was in several ways interesting. To begin with, the current struck where the south end of the Hind Dam had been, and was there deflected sharply, resulting in very serious erosion. Few people who saw the break at this stage believed it possible to hold this erosion from going entirely through the structure, but by unloading immense quantities of rip-rap, the fill was held, the water in front cutting to a depth of about 42 ft.

Two trestles were decided on and started, five pile-drivers being used, one at each end of each trestle and a floating machine in the

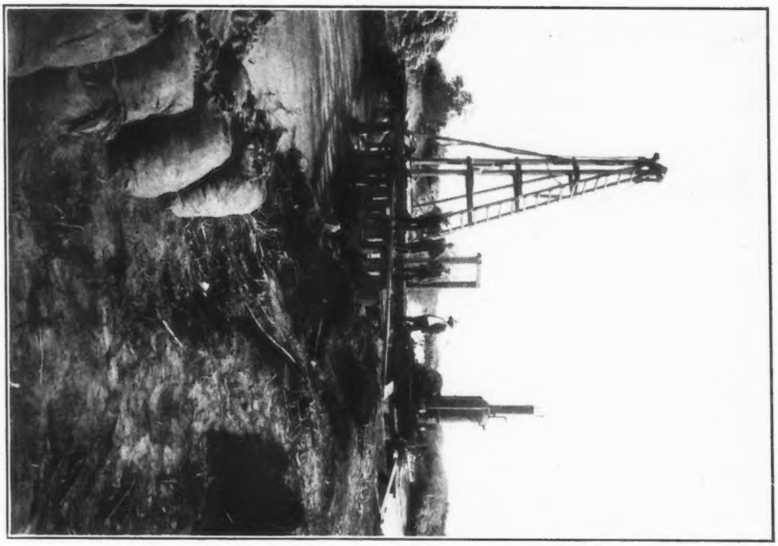


FIG. 1.—CLOSING THE BY-PASS AROUND ALAMO WASTE-GATE. DISCHARGE, 2 500 SECOND-FEET. FINAL HEAD, 10 FEET.

FIG. 2.—RECONSTRUCTION OF LEVEES, SHOWING MUCK-DITCH.

middle of the stream. These trestles had to be thrown in a curve concave up stream to connect with the levee on the south side, therefore the piles had to be driven in a very strong quartering current. The channel was narrowed and the bottom cut down to a maximum depth of 38 ft. In driving 90-ft. piles under these conditions, there was constant danger of overturning the driver and losing the machinery, therefore two complete pile-driving outfits were kept in reserve and two boats waited below the trestle to pick up any men who might be thrown overboard.

On December 28th one line of trestle was practically completed, when a flood, shown on Plate CIX (which, by the way, let the stranded steamer *Searchlight* get back up the river to the work), carrying unusual drift, tore out about one-third of it. Three times this occurred, resulting in the loss of a large quantity of bridge material. All this was obtained through Mr. R. H. Ingram, General Superintendent, Southern District, Southern Pacific Company, and the following telegram from him is interesting:

"Los Angeles, 1/14/07.

"H. T. C.—

"We have exhausted all available supply of piles in San Diego and Southern California. There is very little hope of getting any in Northern California. If you feel that you will need any more please let me know at once as we must make arrangements with the Atlantic System.*

"R. H. INGRAM."

On January 26th, 1907, the first trestle was finished for the fourth time, all stringers were in place, and the track was two-thirds laid. In the second trestle, 50 ft. above the first, seventeen bents remained to be driven. The fill on the south, connecting the trestle with the levee, was 60% completed. Stored on the branch line in Mexico and the United States there were 175 "battleships" of rock, loaded at the quarry of the C. D. Co. by steam shovels, and 100 flatcars of large rock from the distant quarries. At each end of the dam, ½ mile of the levees had been reconstructed, and 1½ miles more opened up, while there were 1 100 men and 1 000 head of stock engaged at Andrade and on the river.

On January 27th, the first trestle was completed, and dumping rock from it began at 5 P. M. By daylight 145 "battleships," containing

*The lines from New Orleans to El Paso.

6 600 cu. yd. of rock had been unloaded as another flood from the Gila began arriving, but it caused little trouble. On February 10th at 11 P. M. the second break was closed, and all the water was again going down the old channel.

The following materials were used in the second closing:

4 000 ft. of railroad trestle—of this, 1 800 ft. were carried away by floods before either trestle could be completed or any rock could be dumped. When rock dumping began, no more trestle was lost; the final result was two trestles, 50 ft. apart, and each 1 100 ft. long, or 2 200 ft. of trestle.

16 000 ft. of 8 by 17-in. Oregon pine stringers—8 000 ft. of these were removed.

1 200 piles.

45 000 cu. yd. of earth placed by teams—making 960 ft. of earth dam to connect with the levee, 31 000 cu. yd. being placed by February 2d.

2 157 car loads, or 55 000 cu. yd., of rock used prior to actually closing off the water.

221 car loads, or 7 735 cu. yd., of gravel.

203 " " " 8 840 " " " clay.

The discharge of the river when work began on December 20th, 1906, was 12 500 sec-ft.

Dec 31st	48 900	sec-ft.
Jan. 7th	15 200	"
Jan. 12th	44 300	"
Jan. 18th	16 300	"
Jan. 20th	33 400	"
Jan. 27th (when first rock dumping began)	13 800	"
Feb. 3d	31 300*	"
Feb. 7th	17 700	"
Feb. 11th	20 800	"

After the break was closed, 956 car loads, or 38 240 cu. yd., of clay and 873 car loads, or 33 555 cu. yd., of gravel were used to complete the Hind-Clarke Dam. The rate of dumping rock, etc., is shown by Table 14.

* 4 300 sec-ft. down the old channel.

FIG. 1.—KINK IN LOWER TRESTLE CAUSED BY DRIFT BREACHING AND TAKING OUT FOUR BENTS OF UPPER TRESTLE.

FIG. 2.—GRAVEL SPREADER IN OPERATION.

TABLE 14.—CAR LOADS OF MATERIAL.

	"Battle-ship."	Flatcar.	Clay.	Gravel.
January 27th to February 2d	606	501	8	15
February 3d to 9th	489	285	58	140
February 10th to 10th midnight	85	91	38	28
	1 180	977	104	173
February 11th to 15th	323	77	99	48
	1 503	1 054	203	221

The water was shut off on February 10th at 11 P. M. and the flatcar rock then on hand and *en route* was unloaded in order to release the cars, the rock from Andrade being used until the 18th in raising the dam hurriedly to protect it against possible floods. The total number of car loads of material used in the closing and in the finishing up of the Clarke Dam were:

"Battleship."	Flatcar.	Clay.	Gravel.
1 490	1 182	784	873

During the entire operations subsequent to the second break, including finishing up the Hind-Clarke Dam complete, but exclusive of the rip-rap to hold the grade before the completion of the trestle over the break on January 27th, the following car loads of material were used:

"Battleship."	Flatcar.	Clay.	Gravel.
1 517	1 240	956	2 052

The clay pit was closed on March 3d, and the Hind-Clarke Dam was finished, except for the final surfacing with the spreaders, on March 15th.

Two steam shovels, and part of the time three (not including the one in the Mammoth gravel pit) were engaged from September, 1906, to May, 1907, at $7.50 per day, prior to January 1st, 1907, and at $12.50 thereafter. During the period, there were from 1 to 12 locomotives, at the following rates:

American, light	$ 8.00	per day.
Moguls	10.00	"
Consolidation	12.00	"
Car pile-drivers, working 40 days at	10.00	"
Donkey engines	1.50	"
Skid-drivers, complete	2.50	"

TABLE 15.—SOUTHERN PACIFIC EQUIPMENT USED ON RIVER, FROM DECEMBER, 1906, TO JULY, 1907, EXPRESSED IN CAR-DAYS.

Month.	Ballast cars, at 25 cents per day.	Steel side dump cars at 50 cents per day.	Cabooses, at 50 cents per day.	Water cars, at 50 cents per day.	Outfit cars, at 30 cents per day.	Home freight car detention (box and flat), at 15 cents per day.	Foreign freight equipment detention.
December, 1906...	595	3 271	80	141	42	371	306
January, 1907....	690	4 972	30	174	....	675	600
February, 1907...	219	4 609	15	153	....	47	78
March, 1907.......	54	5 565	27	321	....	47	71
April, 1907........	....	5 255	4	323	....	28	2
May, 1907.........	2	4 792	9	440	....	17	2
June, 1907........	....	2 635	53	264	....	32	....
July, 1907.........	....	1 336	12	24	....	....	....
Totals......	1 560	32 435	230	1 840	42	1 217	1 059

The Clarke Dam extends from the south end of the Hind Dam across the second break to the levees on the south. These two dams, consequently, constitute one continuous structure, which is known as the Hind-Clarke Dam.

RESULTS OF EXPERIENCE IN CONSTRUCTING THE HIND-CLARKE DAM.

The experience obtained in making these two closings, according to the methods used, afforded some information regarding work of this class which is believed to be entirely unique and in some respects unexpected. In the first place, it was shown that the brush mattress bottom protection is not only unnecessary, but adds to the cost, both in time and money, provided rock is thrown in at a reasonably rapid rate. In discussing the possibility of handling the situation along the lines decided on, the opinion was freely expressed that the rapid current would carry smaller rocks indefinite distances down stream, and that the larger ones would quickly settle into the soft, water-soaked, alluvial soil to indeterminate but very great depths. As a matter of fact, a relatively small quantity of "battleship" rock sufficed to blanket the bottom of the stream with a mattress of rock which fulfilled essentially the same function as a mattress of brush.

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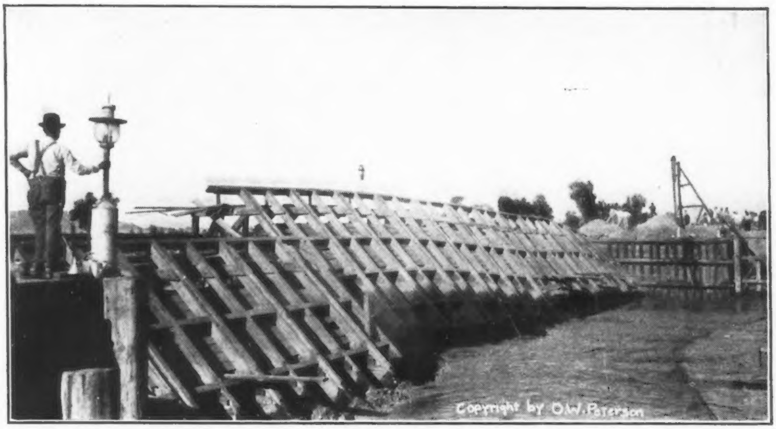


FIG. 1.—FAILURE OF ROCKWOOD GATE, OCTOBER 11TH, 1906.

FIG. 2.—MUCK-DITCH CONSTRUCTION IN LEVEE EXTENSION WORK, 1907.

not absolutely necessary in raising short sections where, through carelessness or a little unexpected settling, an unusual quantity of water is going over a low place in the dam with consequent menacing velocity. Such large rocks were unloaded by a great number of men using pinch-bars, and, to prevent upsetting, the cars were chained to the stringers when unloading especially heavy rocks. One man was killed during each closing by going overboard with a large rock, and these were the only serious accidents. No equipment was lost during either closing.

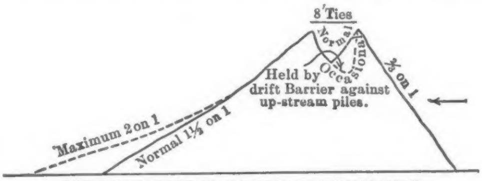
Two trestles were used in both cases. These were 30 ft. apart in the Hind Dam, and 50 ft. apart in the Clarke Dam, the idea being that the current would prevent building a rock mattress extending far enough up stream by merely dumping rock from the trestle on which the closing was made. Careful examination of the resulting cross-section, when the water was shut off the first time, seemed to indicate pretty clearly that it was a needless precaution to have two trestles for final heads of 14 ft. or less. As there was to be no mattress in the second closing, it was decided to use two trestles, in order to take no avoidable chances. Everything worked so well that it seemed safe to do a little experimenting, and, practically speaking, the second break was closed from the lower trestle alone.

In both cases the fills at both ends were kept well above the possibility of being overtopped, and of uniform heights across the remaining length. Train loads extending entirely over the trestles were unloaded most of the time; but short sections of the fill which were low were promptly filled in, and great care was exercised to distribute the overpour evenly. Once, in the building of each of these structures, a local settlement occurred, resulting of course, in large quantities of water going over the relatively short lowered section. The same experience was had in making the upper coffer-dam wall for completing the gap in the Laguna Weir. The construction of a rock fill barrier dam was regarded as impracticable by engineers, because of just such occurrences surely breaching the barrier hopelessly. In point of fact, continued dumping of even small rock soon stops such so-called breaks, although that results in the waste of much material.

The chief difficulty is caused by drift in the stream being caught by the trestle. Theoretically, such drift should be very easily broken up and carried to the bottom by dumping rock. In practice, however,

there is difficulty in keeping drift accumulations from seriously threatening the trestle. In Fig. 1, Plate CXXI, a very sharp kink will be observed which is due to drift accumulations throwing the entire trestle out of line. Two or three bents were taken out of the upper trestle from this cause.

It was found that a cross-section of the dam under the trestle is about as shown in Fig. 22, so that it is the inevitable little breaks and slides, rather than actual settling, which occurs in such cases. Local readjustments or settlings were easily handled with an estimated quantity of 100 sec-ft. per lin. ft. of dam going over the rock fill. How much more could be handled safely with the rock available is not known. The writer would say that experience along the Colorado River would not justify using the rock fill barrier dam method for quantities in excess of half that flow, on an average, although possibly much greater rates of discharge over such a structure might be safe. The quantity which went over the 680-ft. crest was 27 000 sec-ft.



RESULTING SECTION OF RCCK FILL DAMS MADE DURING FIRST AND SECOND CLOSINGS

FIG. 22.

The total length of rock fill dam used in the trestle was 1 125 ft., and the water was allowed to pour over 680 ft. of it; the trestle consisted of 4-pile bents, all piling vertical, 15-ft. spacing between bents, average penetration of piling about 18 ft. The total head, or the difference in water surface above and below the dam, was 12.45 ft., all developed in place in 13 days and 5 hours.

By way of comparison, the building of the upper and lower sides of the coffer-dam within which the central or channel portion of the Laguna Weir was completed is most interesting. One line of trestles, 740 ft. long, was thrown across the stream just above the upper edge of the weir, and another trestle, 728 ft. long, below the lower edge, the trestles being about 400 ft. apart. From these trestles rock fills were built up, and all the water in the river was forced through the sluice-gates at the ends of the weir, the Arizona sluice-way being 2 700 ft. distant and the California sluice-way 1 500 ft. There were well-developed quarries at each end of the weir, and the haul was from 1 500 to 3 500 ft. The following equipment was used:

One 2½-cu. yd. Atlantic type steam shovel (in California quarries).
One 2½-cu. yd. Marion steam shovel (also in California quarries).
Seven 12½-in. American steam hoists, mostly in the Arizona quarries.
Six 18-ton standard gauge Davenport locomotives.
Fifty 6-cu. yd. dump cars.
Eighteen 5-cu. yd. steel dump cars.
Fifty flatcars.

By December 10th, openings to and from the sluice-ways were completed, and the rock fills were advanced until the head was 2.8 ft. On December 19th, the head had been increased to 6.9 ft., of which about 3.5 ft. were on the lower rock fill, while 8 500 sec-ft. were going over these and 3 200 sec-ft. through the sluice-ways. On account of a reported flood, due to heavy rains on the Little Colorado water-shed, work was concentrated on the upper coffer-dam, and by noon of December 21st the gap was closed and the river—9 950 sec-ft.—was turned through the sluice-ways, the head being 3.5 ft. on the lower and 5.4 ft. on the upper fill, or a total head of 8.9 ft.

The river began to rise 30 hours later, and crested at daybreak on the 23d, the discharge being 35 250 sec-ft. Water passed over the entire length—4 200 ft.—of the completed dam, 3.2 ft. deep, but lacked 10 in. of reaching the general level of the upper rock fill. The lower coffer-dam was injured considerably by water running lengthwise of the dam on the down-stream side, so that the upper rock fill stood all the head, which reached a maximum of 10.1 ft. Two weeks later the damage sustained by the railroad tracks on top of the finished parts of the structure had been repaired, the lower coffer-dam had been completed above the danger line, and work had commenced on the 650 ft. of gap in the main structure within the coffer-dam.

About 600 men were engaged on the closing, working in two 10-hour shifts. The total cost of turning the river through the end sluice-gates and constructing the coffer-dam is given as $86 072; the total rock used was 59 750 cu. yd. of excavation, making 82 800 cu. yd. in the rock fill. No lives and no equipment were lost, and the time required after beginning to dump rock was about 2 weeks.

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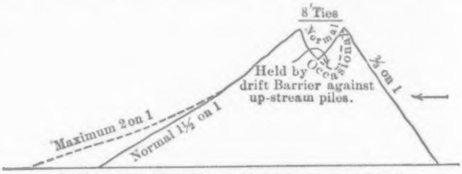

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head charges of the project was $1.04 per cu. yd., of which 45%, or $38 720, was for excavation, Class 1 (quarrying solid rock).

Completing the Clarke Dam.

The Clarke Dam was rendered impervious by dumping Mammoth pit gravel and clay from both trestles. No attempt was made to puddle or settle the material by hydraulicking, as was done with the Hind Dam. Small local settlements occurred from time to time for a year after the dam was completed, but nothing disquieting in any way. Imperviousness was very quickly obtained, indeed, long before the structure was raised to grade and widened to its proper dimensions.

In constructing the Hind Dam, every effort was made to hurry the work, and the stringers were not taken out. With the Clarke Dam, all stringers were removed, the tracks were raised and narrowed to 13 ft. from center to center, and the top and sides were finished off with the gravel spreader used on the levee work, leaving the finished structure as shown on Plate CXIII and Fig. 1, Plate CXXIV.

Spur Levee.—A spur levee, 8 700 ft. long, was built, starting at the elevation of the top of the Hind-Clarke Dam (124 ft.), and with an initial descending grade of 0.5 per cent. The purpose of this levee is to prevent water from any break in the main levee south of the Hind-Clarke Dam from getting into the old dry channel below, as happened when the second break occurred. It is intended to hold such water back and make it spread over the low country in a sheet. This levee, which was decided on, but not started, before the second break, is located along an old overflow channel slightly higher than the country on either side, and where test-pits showed need for little muck-ditching. Arrangements were made later with the Inter-California Railroad to change its alignment to use this spur levee and the Hind-Clarke Dam, and it was thus extended by this railroad grade for 4 miles without any opening.

Changes in Staff.

The success of the Hind-Clarke Dam being assured, arrangements were concluded whereby F. C. Herrmann, M. Am. Soc. C. E., on February 1st, 1907, became Assistant Chief Engineer of both the C. D. and Mexican Cos., particularly for the purpose of making surveys and estimates for reconstructing and extending the canal system in Im-

perial Valley. About March 1st, the Clarke Dam being then well advanced, Mr. Clarke was transferred to the valley and appointed Superintendent of Construction of both companies, Superintendent Hind remaining in charge of all operations along the river until they were finished, when he was transferred to the Harriman Lines in Mexico, on August 1st, 1907. On May 1st, Mr. Clarke returned to the railroad as Resident Engineer of the Coast Division. Nearly two years later he came back to the valley as Superintendent of Imperial Water Company No. 1, and early in January, 1910, was appointed by the Receiver of the C. D. Co. as Assistant General Manager and Chief Engineer, which position he resigned in April, 1911. The writer relinquished the title of Chief Engineer in both companies in July, 1908, and issued circulars advancing Mr. Herrmann to the positions which he held until he left the companies, in March, 1910, after the appointment of a receiver. His successor in the Mexican Co., and its present Chief Engineer, is Mr. C. N. Perry, who first came to the valley with Mr. Rockwood in 1892, and was Resident Engineer of both companies from October, 1901, to August, 1906, and so was in immediate charge of most of the existing canal construction in Imperial Valley. Since April, 1911, Mr. J. C. Allison has been Chief Engineer of the C. D. Co.

Levee Reconstruction.

On December 20th, 1906, reconstruction on the existing levees was begun by tearing away the land side of the levee and excavating a continuous muck-ditch as deep as the test-pits indicated to be necessary. The usual location of a muck-ditch is under the center of the levee section, but, in reconstruction work, this was not practicable, and besides, there were several reasons for location nearer the land toe, which will be mentioned later. The excavation was made as narrow as possible with the use of 4-horse Fresno scrapers, and it was found that the walls, not only of the natural soil, but of the recently constructed levee section, stood practically vertical without any caving. (Fig. 2, Plate CXX.) The muck-ditch was excavated 1 ft. lower than the lowest layer of cracked adobe soil lying above the permanent water-table, and was refilled with the material removed, care being taken to keep out roots and clods of adobe exceeding 3 or 4 in. in greatest dimension. When the muck-ditch was completed, the land side of the levee was replaced to the slope of 3 to 1, instead of 2 to 1, as origin-

ally built. This work was started on force account, but was soon changed to a yardage basis, on the following schedule:

Levee section removed and replaced in embankment, 12 cents per cu. yd.

Muck-ditch excavation and refill, $17\frac{1}{2}$ cents per cu. yd.

Reinforcing levee section, 19 cents per cu. yd.

The total earth handled was 199 000 cu. yd.

Levee Extension.—At the same time, surveys were commenced for extensions of the levee to the south. From Mile 7 the original alignment continued closely paralleling the river, and here all clearing had been done for 2 miles, and about 20% of the fill had been made. At the south end the flood caused three breaks, close together, through almost completed levee section. These breaks were due to bad material, that is, adobe which in working had broken up into small, hard clods. Directly across the river, similar breaks had occurred in the levee of the Yuma Project. In addition, the river during this flood showed a marked tendency to cut into the west bank immediately opposite.

The surveying party found that the most suitable soil and the highest ground, west of the river bank itself, lay along the Paredones overflow channel, which turned away more than a safe distance from the river. This channel was followed beyond the undergrowth to open country, and a hurried reconnaissance showed that at some future time, by building approximately 20 miles of levee, most of it relatively low, connection could be made with the mountain chain on the west side of the delta at Cerro Prieto.

It was felt that the work now in progress was not merely to prevent flood-waters from quickly getting around the end of the Hind-Clarke Dam, but that very soon a system of dikes constituting a continuous line of defence for the Imperial Valley must be provided. President Roosevelt had not requested or authorized Mr. Harriman to construct anything so elaborate, but only what would form a protection for a few years at most, or until suitable permanent arrangements could be made to control the river properly. Such levees as were to be built, however, obviously should be constructed with a view to being incorporated in their entirety in the final scheme.

Two plans for keeping the overflow waters of the Colorado from getting into the Salton Sea are at once seen to be better than any

others. One is a levee line along the Paredones ridge and north of Volcano Lake to Cerro Prieto, and the other is a line of levees parallel to the river practically to tide-water. These two plans are shown in Fig. 4. The first would protect all American interests and all that territory in Mexico lying to the north and west; the second would also protect the very large area lying between. This additional area is the property of the C. M. Co., and is in an extremely good location for irrigation from the Colorado. The C. M. Co., on being approached regarding the matter, thought that its present interests would be best served by the northern line of defence, which thus precluded the possibility of financial assistance from that company in giving additional protection, and the matter resolved itself into a decision as to which of the two lines could be built and maintained at least cost. The instrumental data regarding the region were insufficient to decide between the two possibilities, and, incidentally, the same condition of affairs unfortunately exists to-day. However, something had to be done, and at once. Based solely on careful reconnaissance, the northern line of levees was decided on, and the portion to be immediately built was located to a point about 10 miles below the Hind-Clarke Dam, this being enough to prevent water from getting around into the Alamo channel during flood stages. Aside from the desire to do only what was absolutely required and could certainly be finished before the summer flood, it was felt that the officials of the Mexican and American Governments should have the greatest freedom in determining on future work for controlling the river.

The grade was determined from existing high-water marks along the line of the levee, checked by high-water marks along the line of the river; and several miles at the lower end has a grade of 3 ft. per mile. The cross-section decided on was a top width of 10 ft. and slopes of 3 to 1 on each side.

Muck-Ditches.—The Yuma Project, U. S. Reclamation Service, loaned one of its engineering corps, the method being to grant leave of absence to the members and have them carried on the payrolls of the Mexican Co. This corps was in charge of Mr. C. W. Ozias, Assistant Engineer, and was assigned to levee location work. Mr. Ozias was instructed to make tests for determining the time at which efficient compression of adobe soil would take place with distributed loads, as under a levee when immersed in water. For these tests some hard

chunks of adobe were taken from the muck-ditch excavation, 2 ft. below the surface at Station 178, and placed in a box 1 ft. in each dimension, in as nearly natural condition as possible, and kept under a continuous pressure of 500 lb. per sq. ft. At the end of 1 hour ½ cu. ft. of water was added, part of which leaked out. Compression started rapidly, and continued for 1 hour, due to the chunks being pressed

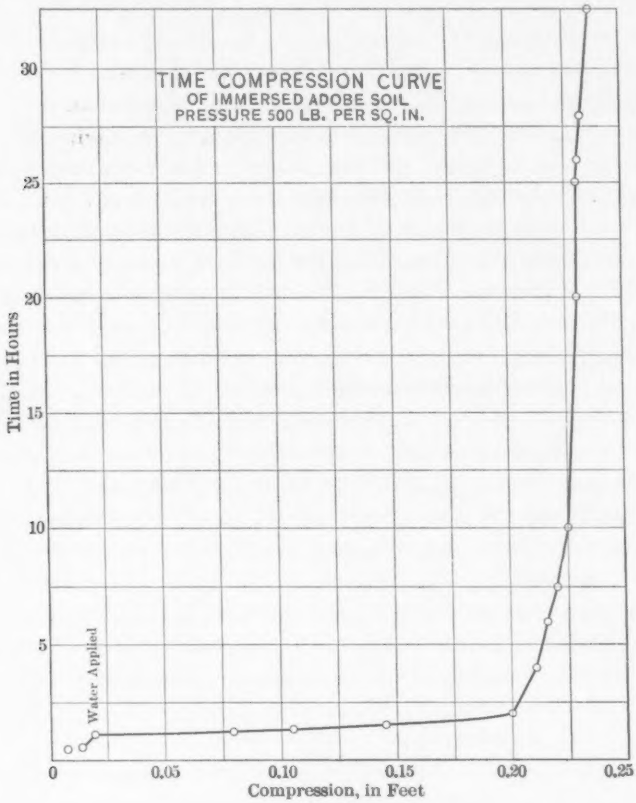


FIG. 23.

closely together in the box, and slower compression started at the end of this time. The compression continually decreased until the 40th hour, when no more movement was noticeable on the scale beam. The mass was then removed from the box and found to be plastic and pressed together so that it contained no voids. From this it appeared that efficient compression under a load of 500 lb. per sq. ft.

starts at the end of an immersion of about 3 hours, and continues indefinitely until all the voids close. This is why borders, canal banks, levees, etc., in that region soon become impervious if breaks can be prevented during that period. It is the inability to prevent breaks over a long line which renders a muck-ditch of some form practically necessary.

The results of this experiment suggested dampening the slickens under the levee section enough to have the voids in it closed by compression due to the weight of the levee section. Such wetting could be brought about by digging a trench along one or both toes of the levee and pumping water into it, and would save the large cost of muck-ditch construction. As this levee line was too vitally important for such an innovation, it was decided to use a muck-ditch 6 ft. wide on the bottom and having side slopes of 2 to 1, the excavation to extend at least 1 ft. below the lowest layer of cracked adobe soil above the water-table. Test-pits were dug every 200 ft. to determine the necessary depth.

The muck-ditch, however, was located under the land toe of the levee, as shown in Fig. 19, because it was not deemed desirable to prevent water from getting under the levee, but, on the other hand, to allow water to go under it as far as possible and still certainly hold it back with the muck-ditch. In this way the cracked adobe layers under the levee section would become impervious as a whole in the minimum time after the water came as high as the toe of the levee.

The muck-ditch, except for two short stretches, was filled with the soil taken from it, only a few places being found in the 17 miles constructed where bad material occurred in such masses as to render it at all difficult to mix the adobe clods with satisfactory material taken out of the excavation.

Borrow-Pits.—Borrow-pits were located on the land side, leaving a berm of 40 ft. and traverses 20 ft. wide at intervals of 250 ft. It was specified that borrow-pits be left in workmanlike condition, with a maximum depth of 2½ ft. on the side nearest the levees and 4 ft. on the farther side. The reasons for borrow-pits on the land side have already been given. The occurrence of the second break was not considered to indicate any good reason for change in this particular.

Contracts and Contractors.—All levee extension work was done on a yardage basis, a contract being let to W. K. Peasley and Company, of

Los Angeles, for 18½ cents per cu. yd., for the entire levee cross-section, including the muck-ditch, and 10 cents per cu. yd. for refilling the muck-ditch section. The price for clearing and grubbing was $200 per mile. The contractor assumed all risk of expenses due to delays caused by flood, and the company paid all customs charges at the Boundary Line.

The Yuma Project, U. S. Reclamation Service, permitted the use of 120 head of its rented stock in order to assist in completing the work before the summer flood. This outfit worked under the usual regulations of the Reclamation Service, on a rental basis, and the levee contractor put it on the work and paid the Mexican Co. what the latter paid. The Reclamation Service engineers advise that levee construction on the Yuma Project has been found to cost much less than Mr. Peasley's contract, but, for some reason or other, the levee contractor and the company's timekeepers, who checked the work, show that the contractor's price for the work done by this grading outfit on this job was less than its cost. The great hurry had a marked effect on the expense to the contractor, however. At any rate, the writer had an opportunity to examine the contractor's books later, and found that his profit on this job, with proper overhead and equipment deterioration charges, was 17.3%, and none of the possible delays due to floods, etc., occurred.

All work was very carefully inspected by an unusually large corps of men, and every precaution was taken to prevent any roots or rubbish from getting into muck-ditches or fills, and to see that all muck-ditch construction was carried to the depths ordered.

Railroad Track.—When completed, all levees, both as reconstructed and extended, were laid with a railroad track consisting of new 6 by 8-in. Oregon pine ties 8 ft. long and good old 56-lb. relaying steel obtained from the Southern Pacific Company. This was done partly because of the great advantage in maintenance work in future and partly to distribute a blanketing of Mammoth pit gravel.

On the completed work 15 miles of track were laid on the main levee, 1½ miles on the spur levee, 5¾ miles from Hanlon's Junction to the Lower Heading, 2.6 miles of sidings, quarry tracks, etc., in California, and 2.7 miles of sidings and double track over the Hind-Clarke Dam in Mexico. No part of this track has been taken up, but the main

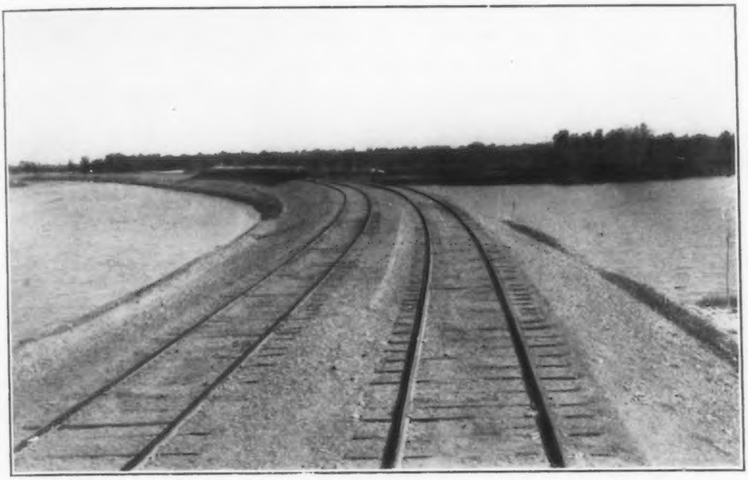


FIG. 1.—SOUTH END OF HIND-CLARKE DAM, SHOWING LOCATION OF SECOND CLOSING.

FIG. 2.—TYPICAL SECTION OF COMPLETED LEVEE WITH LAND-SIDE BORROW-PITS.

line and sidings between Hanlon's Junction and the end of the spur levee have been sold to the Inter-California Railroad.

Gravel Blanketing.—Vegetation, springing up like mushrooms on fills in the region, very soon precludes any possibility of inspection. The roots also attract burrowing animals. Furthermore, the danger of erosion on the water face by swift currents due to the fall of 3 ft. per mile made it exceedingly desirable to provide a better surface for the eddy currents. For these reasons all levees were blanketed on the top and both sides with a 15-in. layer of gravel from the Mammoth gravel pit, which, as has been said, supplies a cementing material which packs into an almost impermeable surface. It was practically impossible—and would have cost more money—to have blanketed the water face only or to have put a greater depth on it than on the land side, because all the gravel was hauled in "battleships" which dumped equal quantities of material on each side. Ordinarily, two cuts of cars were unloaded, a considerable portion of the first being used on the top and to surface the track. The remainder was spread evenly over the two slopes of the levee with a home-made spreader devised and constructed by Superintendent Hind at a total cost of $300. Its construction is shown on Fig. 24 and Fig. 2, Plate CXXI. With this the gravel was spread in an extraordinarily workmanlike manner at a cost of 0.1 cent per cu. yd. It worked in either direction, and the ordinary process was to have one cut of gravel dumped along from 3 000 to 6 000 ft. of levee, and have a locomotive take the spreader on two round trips, the usual speed being from 10 to 12 miles per hour.

Very much more permanent and expensive blanketing of levees is to be found on the Sacramento River, California. In Reclamation District 307, near Lisbon, about 15 miles below Sacramento, a length of a little less than 3 miles of levee has recently been completed. This keeps out the back waters, in which there is little current, but, on account of the width, the wave action is relatively severe. The blanketing was begun about July 1st, and finished about December 1st, 1911, and consists of 700 000 sq. ft. of reinforced concrete 4 in. thick. (Fig. 2, Plate CXXV.) The reinforcement was a No. 10 gauge wire, 6 by 6-in. mesh, known as the Clinton Electrically Welded Fabric. The concrete was a 1 : 2 : 5 mixture, with rock brought by train from Oroville to Sacramento and thence by barge to the work, and cost there about $1.50 per cu. yd. The Reclamation District furnished the cement

and reinforcing material, and the contractors, Richard Keating and Sons, of San Francisco, did all the other work. Mr. P. N. Ashley, County Surveyor of Yolo County, Woodland, was engineer. This covering cost 13 cents per sq. ft., or a total of about $94 000, or approximately $32 000 per mile. The average height of the levee was 22 ft.

TABLE 16.—STATEMENT OF CHARGES ON ACCOUNT OF OPERATION OF MAMMOTH GRAVEL PIT, FROM OCTOBER 15TH, 1906, TO JULY 15TH, 1907, INCLUSIVE.

Labor of gangs in pit getting out gravel		$23 312.46
Use of tools, 2% on above labor		466.25
Wages of trainmen in pit		3 108.91
Wages of enginemen in pit		2 111.65
Wages of operators, etc., at pit		897.19
Rental of engine in pit		2 202.50
Rental of steam shovel in pit		2 457.50
Fuel furnished for pit engine		4 495.14
Fuel furnished for steam shovel		1 594.41
Material purchased for pit engine repair	$184.63	
Material purchased for steam shovel	548.90	
Miscellaneous supplies	6 890.88	
Miscellaneous pit engine supplies	237.98	
		7 862.39
On store department expense, 5%		393.10
Shop repairs to steam shovel		727.19
On shop expense, 10%		72.72
Freight on material shipped to pit		6 345.86
Miscellaneous credit		*119.74*
Total		$55 927.53

TABLE 17.—CARLOADS OF GRAVEL SHIPPED.

Month.	To California Development Company.	To Epes Randolph, Agent, S. P. Co.	To Southern Pacific Company.	Totals.
1906.				
October	357	.....	....	357
November	1 171	.....	18	1 189
December	.....	593	718	1 311
1907.				
January	.....	479	644	1 123
February	.....	756	726	1 482
March	.....	1 849	300	2 149
April	.....	2 178	114	2 292
May	.....	2 032	233	2 263
June	.....	1 082	527	1 609
July	.....	239	59	298
Total	1 528	9 206	3 339	14 073

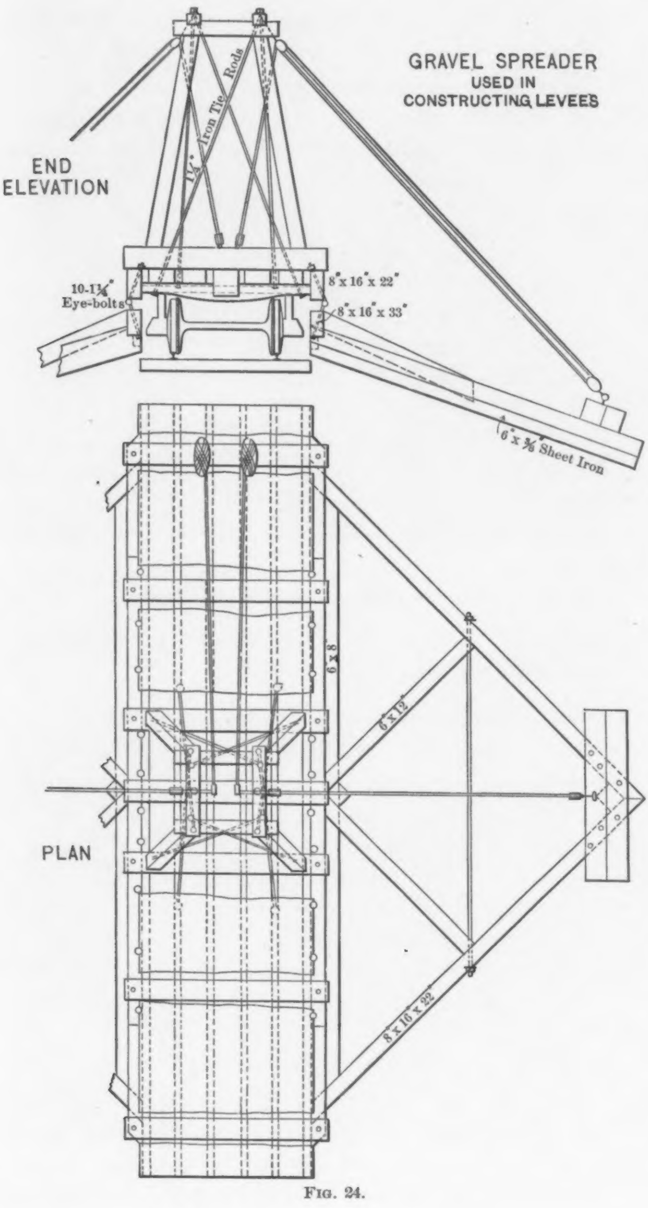

GRAVEL SPREADER
USED IN
CONSTRUCTING LEVEES
END
ELEVATION
1¼" Iron Tie Rods
10-1¼"
Eye-bolts
8"x 16"x 22"
8"x 16"x 33"
6"x ⅜ Sheet Iron
PLAN
6 x 8
6"x 12"
8"x 16"x 22"

FIG. 24.

The following is a detail of charges for royalty to Epes Randolph, Agent, S. P. Co.:

Value of tracks in Mammoth gravel pit, property of S. P. Co.		$16 832.00
Interest on $16 832.00 for 1 month at 6% per annum	$75.74	
Depreciation on $16 832.00 for 1 month at 5% per annum	70.13	
	$145.87	
Interest from December 1st, 1906, to July 15th, 1907, 7½ months		1 094.03
Total number of cars removed by Epes Randolph, Agent, S. P. Co., 9 206, or	$0.11883	interest per car.
Charge for gravel, 1 cent per cu. yd., 30 cu. yd. per car, or	0.30	royalty per car
Total royalty charged per car	$0.41883	

Average cost per car, $55 927.53 divided by 14 073 = $3.9741014 per car.

In blanketing the levees 5 285 carloads, or 185 000 cu. yd., were used. Of this, 4 803 carloads, or 168 000 cu. yd., were used on the 15 miles of main levees, or 16 800 cu. yd. per mile, and 482 carloads, or 17 000 cu. yd., on the 1.6 miles of spur levee, or 10 633 cu. yd. per mile.

Wing Levees.—In addition to blanketing the levees with cementing gravel, wing levees of gravel were built out into the brush, at intervals of about 400 ft. Their purpose is to check the flow of water in the V-shaped section between the trees along the near toe and the water face of the levee. Brush abatis work was not considered sufficiently permanent.

Telephone Line.—A two-wire, metallic-circuit, telephone line was constructed along the land toe of the levee after the gravel blanketing was finished. The location was unfortunate, as the rapid growth of willows and other bulb vegetation just behind the levee caused considerable annoyance and expense. It would have been much better

FIG. 1.—TYPICAL SECTION OF COMPLETED LEVEE, WITH INTERRUPTED BORROW-PITS ON THE RIVER SIDE.

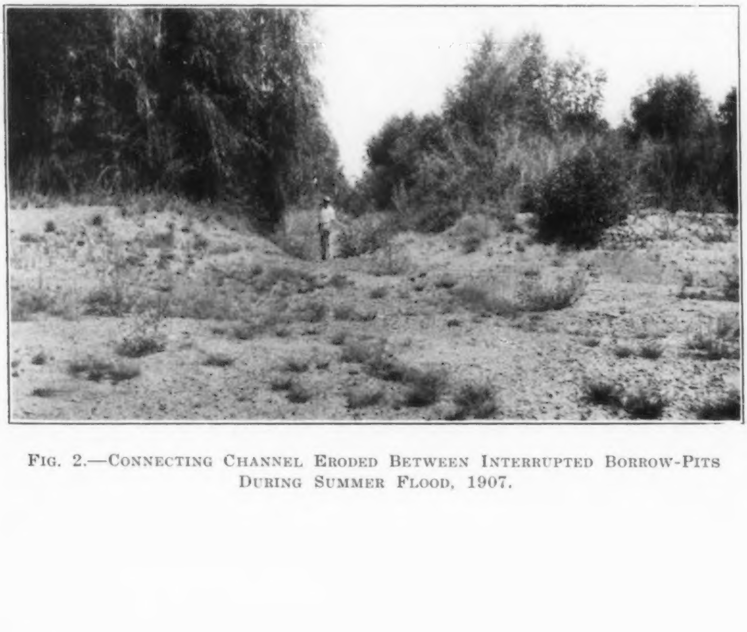


FIG. 2.—CONNECTING CHANNEL ERODED BETWEEN INTERRUPTED BORROW-PITS DURING SUMMER FLOOD, 1907.

to have put the line on the land slope, about 9 ft. from the center of the levee.

At each mile post a 6 by 8-ft. wooden tool and telephone house was put up, and white boards were nailed to telephone poles at points half way between, to mark the ends of the patrol beats. When floods are expected, sacks, tools, etc., can be distributed and kept in these houses, but between times it was found impracticable to leave anything but the telephones on account of malicious and thieving passers-by. It was also found that the heat in these houses during the summer was sufficient to melt down and spoil many of the rubber receivers, so that only metallic ones are now used.

General Data.—The total length when complete was 15 miles of main levee, having an average height of 8 ft. and an average muck-ditch yardage of from one-third to one-fourth of the main section, and 1.6 miles of spur levee, with a height of from 20 to 4 ft. and a muck-ditch yardage of about 15% of the main section. The quantities and costs of the extension work after the second break were as follows:

154 500	cu. yd.	muck-ditch	excavation at $18\frac{1}{2}$ cents.
49 700	" "	" "	refilling, which was twice handled, at 10 cents.
443 000	" "	embankment at from $18\frac{1}{2}$ to $25\frac{3}{4}$ cents.	
166 700	" "	original specification at $18\frac{1}{2}$ cents.	
100 800	" "	wide check at 19 cents.	
154 545	" "	checkerboard system at $25\frac{3}{4}$ cents.	

9.66 miles of clearing and grubbing, $2 000.

Enlarging Main Canal and Building Secondary Levee on Canal Bank.—On November 26th, 1906, the *Delta* cut its way into the Main Canal just below the Boundary Line and commenced deepening it and building a secondary levee on its river side beginning at the concrete head-gate and working down stream. It reached the Lower Heading and connected the levee bank with the north end of the Hind Dam in May, 1907, and started back, improving the work at various places and continuing this until July 21st, 1907. This levee was built in exactly the same way as those around the Reclamation Districts of the Sacramento River, and was made with a minimum height of 3 ft. greater than the main levee opposite along the river bank. While not a very efficient construction, it,

nevertheless, will serve to prevent water from getting into the Main Canal from breaks in the main levee until they can be repaired.

Criticism of Levee Work Done.—All parties interested assumed that the Harriman interests were doing work in the capacity of a contractor for the United States Government, and not in any sense in its own behalf, and that the engineers of the Reclamation Service were "available for consultation." Certain of these engineers, individually, criticized the excavation of the borrow-pits on the land side of the levee, and so a request was addressed to the U. S. Reclamation Service for a Consulting Board to consider the situation and make recommendations. Such a Board was appointed, all being members of the Service, consisting of A. P. Davis, W. H. Sanders, D. C. Henny, and Francis L. Sellew, Members, Am. Soc. C. E. This Board inspected the works then under way, and on January 10th reported, recommending among other things that:

"4. Borrow-pits should be on the water side, berms between them of greater width than the pits, and care should be taken not to disturb the vegetation on the berms, which should also be protected at frequent intervals with barbed-wire fences of 4 to 5 wires, the bottom of which is at the surface of the ground * * *.

"7. The levees now built should be provided with cut-off trenches under the water slope and later provided with sheet-piling reaching below borrow-pits * * *.

"8. All levees should be blanketed with gravel on water slope and railroad track maintained on the levee."

The clearing for half of the extension work was done at this time, and the work already started was continued as theretofore. Fear of erosion trouble, because of borrow-pits on the river side, and correspondence with the New York office of the Harriman Lines, resulted in the appointment of another Consulting Board consisting of Messrs. L. C. Hill, M. Am. Soc. C. E., W. H. Sanders, and F. L. Sellew, of the U. S. Reclamation Service, and William Hood, M. Am. Soc. C. E., Chief Engineer of the Southern Pacific Company, which met at Yuma, thoroughly examined the work, and reported on February 14th, among other things, as follows:

"*Existing Levee Between Cement Head-Gate and Dam Across Lower Intake.*

"Spur dikes (traverses between borrow-pits) to be increased in width to at least 50 ft. on top and to be at least 4 ft. in height above

the general level of the original surface of the ground, and to extend at least 300 ft. northerly from the northerly edge of the borrow-pits, and in this 300 ft. no borrow to be made on either side of this levee and no brush to be cut outside of the limits of the slope stakes.

"The end of the levee and for some distance on the sides on each side near the end to be thoroughly brushed.

"These cross-dikes to be not to exceed 600 ft. apart, and where now located essentially farther than this, an intermediate cross-dike to be put in * * *.

"An abatis work, being in effect a wire and brush wing dam, shall be built from a point on the slope of the levee nearest the river, situated well above high water, and such wing dam pointing down stream, approximately, per local conditions, at an angle of 45 degrees to the levee.

"This to be made with suitable posts or stakes driven into the levee and between the levee and existing trees and thence by assistance of trees acting as posts or stakes and suitable barbed wire fencing in two lines not less than 2 ft. apart, and the whole filled with brush thoroughly wired down.

"These wing dams to occur at no greater distance apart than 500 ft. * * *.

"Levee Below Dam Across the Lower Intake, This Being Partly Completed and Under Construction.

"The same remarks as to spur dikes and abatis work apply as stated above with reference to existing levee between the cement head-gate and the dam across the lower intake * * *.

"For the levees as constructed, material has been taken from the land side instead of the river side, as recommended by the previous Consulting Board of the Reclamation Service.

"As these conditions now exist, the present recommendations are with a view to make the levees as secure as practicable under present conditions * * * . As to the still unconstructed portion of the $10\frac{1}{4}$ miles of levee now intended to be constructed southwesterly from the dam across the lower or Mexican intake, this unconstructed portion being several miles, we recommend and expect that the recommendations of the Consulting Board of the Reclamation Service as to the position of the borrow-pits; position, depth and character of muck-ditches, and all other matters, be strictly complied with."

Accordingly, the additional levees constituting 4.11 miles at the south end were built with muck-ditch under the river slope and with borrow-pits on the water side of the levee. These pits are 100 ft. lengthwise of the levee, with spaces between them 100 ft. in width, on

which every care was taken not to disturb the vegetation to the water toe of the levee. This method of borrow-pits is known locally as the "checkerboard" system.

It is obvious that the checkerboard system requires an average haul of excavated material of about 175 ft. in excess of that required by the ordinary plan.

According to comparative tests this increase in length of haul and inconvenience in handling teams resulted in increasing the cost of work about $6\frac{3}{4}$ cents per cu. yd. for embankment only, or approximately 30% of the cost of team work.

It was deemed advisable to complete the main levee high enough to be beyond danger of overtopping by floods before doing anything on the spur levees between the land side borrow-pits recommended by the Consulting Board on February 14th. About the middle of May bids were asked for constructing them (described in the first item of the report of the second Consulting Board), and the lowest bid was $40 per acre for clearing, $80 per acre for grubbing, and 31 cents per cu. yd. for embankment, making their estimated cost between $12 000 and $12 500 per mile of main levee. Accordingly, a third Consulting Board was requested, and was appointed by Mr. A. P. Davis, Chief Engineer of the Reclamation Service, consisting of C. E. Grunsky, Consulting Engineer to the Secretary of the Interior; L. C. Hill, and F. L. Sellew, which Board met on June 19th and recommended as follows:

"1. That in lieu of the general system of cross-dikes recommended by the second Consulting Board:

"(a) To complete at once the cross-dikes now being constructed about 500 ft. southerly from the south end of the Clarke dam, making it 10 ft. wide on the top and giving it the same height as the crest of the spur levee with which it connects;

"(b) To construct a second cross-dike about 500 or 600 ft. southerly from the first, also 10 ft. wide on top and with a height the same as the crest of the spur levee with which it connects;

"(c) To construct cross-levees, with crest at least 4 ft. above made ground (tops 10 ft. wide) between the main levee and the secondary levee [along the east bank of the main canal from the concrete headgate to the north end of the Hind Dam. H. T. C.], near or on the southerly bank of the old upper Mexican intake, and a second within about 1 000 ft. of the southern end of the secondary levee."

This recommendation was carried out.

Summer Flood of 1907.

The levees, including the secondary one along the Main Canal and the cross-levees mentioned in the recommendations of the last Consulting Board of the Reclamation Service, were completed on July 21st, 1907, and the Epes Randolph, Agent, fund was closed. The *Delta* continued deepening the Main Canal and at the same time raising and strengthening the secondary levee on its river side for 3 months longer, but it was estimated that the cost of such work balanced the deepening of the Main Canal done while strengthening the same secondary levee, prior to that date. After that time all charges for maintenance and operation of the head-gates and levee system were made by the Mexican Co. against the C. D. Co.

The flood of 1907 was a record one in total discharge, and probably would have been in gauge height reached at Yuma had it not been for the unusual conditions lower on the river. The old channel of the Colorado was considerably higher than usual, from the point of diversion to the Gulf, the erosion since the diversion and until the coming of the summer flood being of not much importance. It had been silted by the very small quantity of water carried as the re-diversion became more and more a reality, and, in addition, it was appreciably raised by the flood of December 5th which stranded quantities of heavy silt.

The vegetation of the lower delta lands depends on the annual overbank flows, and these lands had not been covered for two seasons, due to the river diversion north of the delta's dividing ridge, much of the light vegetation had perished, and large tracts of the region had been burned over. In designing the levee system it was deemed conservative practice not to take into account any such increased overbank flow, and to consider that the whole channel would be much less efficient and would deepen much more slowly than usual under the coming summer flood, on account of the fact that the bed had been undisturbed for 2 years, and hence was compacted and dry. However, the flood of 1907 came up very gradually, and eroded the bed most satisfactorily, and an extraordinary quantity of water went overbank, particularly to the west because of the greatly decreased vegetation. It thus happened that this record flood, confined between levee banks only 1 500 ft. apart, rose to only an average of about 6½ ft. below the top of the levee, varying from a minimum of 6 ft. to a maximum of 7 ft. The water got against

the levees for their whole length, however, testing the muck-ditch construction thoroughly, and in no case was any weakness apparent.

The water-table throughout the region was raised above the bottom of the borrow-pits in many places, so that a considerable quantity of water, in some cases 2½ ft. in depth, slowly seeped into these pits. This water was always perfectly clear, came in very slowly, and gauge readings, kept in many of them and in pits nearby, showed that the levels in them fluctuated with the adjoining water-table at all times. As the flood went down, these waters lowered and disappeared, and a rapid growth of willows started, so that a very large part of the borrow-pits is now overgrown with a dense growth, many bushes attaining a height of 20 ft. in 2 years.

Much difficulty has been found in keeping Indians from clearing away such growths and utilizing the borrow-pits as garden patches. Indeed, the Mexican officials practically take the position that, while the land is private property belonging either to the Mexican Co. or the C. M. Co., the nomadic Indian tribes have for many years been free to live their Gipsy life therein; that the levee system has cut off the annual inundation on which these people depended for their garden crops; and that it is unreasonable to insist on preventing them from taking these borrow-pits for gardening purposes, particularly as they are prevented from making clearings and utilizing inundated land in front of the levee.

A very striking occurrence was the complete filling up of the diversion channel between the river and the Hind-Clarke Dam. In many places this was more than 45 ft. in depth and probably averaged 20 ft. for 2 800 ft. up and down the river and 3 300 ft. at right angles thereto, an area of 210 acres. When this summer flood had passed, there was only a little pool about 5 ft. deep immediately above the Hind-Clarke Dam where the water had been deepest. Two years later this area was so densely overgrown with willows that it was extremely difficult to believe that the break ever occurred there. Until this silting up in front of the Hind-Clarke Dam began, numerous small boils appeared in the sand bar formation behind the Hind Dam. These at no time were of any importance, and soon ceased. Behind the Clarke Dam, practically no seepage whatever occurred.

The very slight percolation of water under these structures was rather surprising and very gratifying, considering their non-homogen-

FIG. 1.—SECOND CLOSING. HEAD DEVELOPED, 8 FEET.

FIG. 2.—REINFORCED CONCRETE FACING, ON LEVEE 25 FEET HIGH, NEAR SACRAMENTO, CAL.

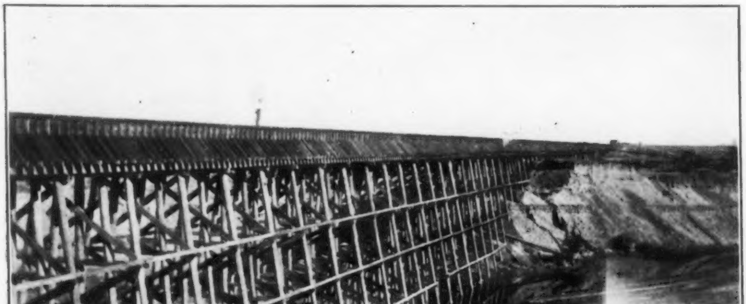


FIG. 3.—WOODEN FLUME OVER NEW RIVER BARRANCA.

eous structure and the character of their foundation. Nevertheless, it accords with the experience with the wooden and concrete head-gate coffer-dams, and the pumping of water into the Main Canal to keep the *Alpha* afloat, namely, that seepage through the alluvial soil of the region is remarkably small.

The water-table, for considerable distances on each side of the river, rises and falls with the water surface, especially during the summer floods, with small lag in time. The writer has always considered these two facts incongruous, and has never been able to find a satisfactory explanation.

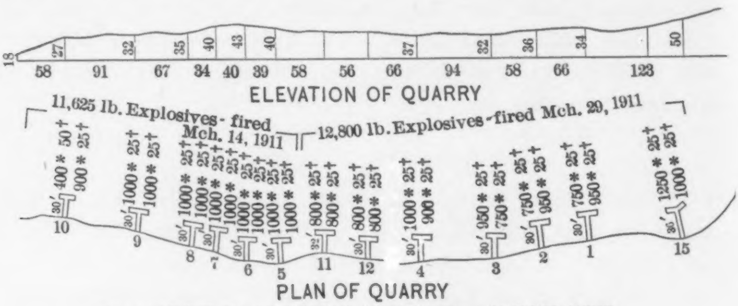
Maintenance of Levees.

Until the summer flood of 1907 had passed, the levees were very carefully patrolled, watchmen being stationed night and day at each telephone house. A considerable store of timber, large and small, has always been kept at the Lower Heading until recently, and a storehouse containing a large quantity of shovels, picks, crow-bars, track tools, and lanterns has been maintained at the upper end of the Hind Dam. In this, from 20 000 to 50 000 sacks are always kept, being drawn on for use in the valley and replenished to avoid depletion. Until a number of flashy floods had passed, a work train was always ordered from the Southern Pacific on reports from the head-waters indicating large rises in the lower river, and sacks were taken from the storehouse and distributed up and down the river, 500 at each toolhouse. Confidence in the effectiveness of the dikes soon grew, so that this is no longer being done.

In spite of the gravel blanketing, vegetation started up slowly, and is kept down by constant but relatively inexpensive work. Considerable annoyance is caused by insects, particularly large burrowing ants which make great ant-hills and holes. These are destroyed by pouring gasoline into the holes and burning it.

As the value of rock, in repairing breaks or in river protection work, to prevent the side cutting of banks from breaching the levees, made it advisable to have a large quantity ready in the Andrade quarry, about 50 000 cu. yd. were blown out ready for handling by steam shovels. This was done by "coyoting" or driving 3-ft. tunnels horizontally about 30 ft. into the foot of the rock face, at intervals of 60 ft., and driving cross-tunnels at the ends, 10 ft. on each side.

These cross-tunnels were loaded with black powder, which was all exploded at one time. All this rock has been taken away, and has left the quarry with a practically vertical face, averaging 50 ft. in height for a length of 1 000 ft., a very satisfactory condition. An American Hoist Company's 10-ton steel derrick with a 60-ft. boom, erected to handle this rock, proved very efficient and satisfactory. The railroad

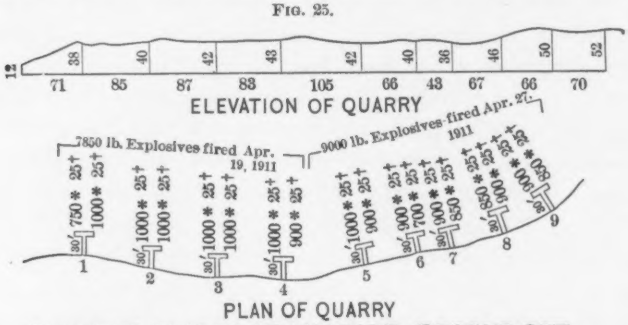


CHAMBER BLASTS AT ANDRADE—FIRST SET

Pounds, Judson Powder ——* Pounds, 40% Giant ——†

Loosened up 51,600 cu. yd. Swell, 10,000 cu. yd. Material broken small

FIG. 25.



CHAMBER BLASTS AT ANDRADE—SECOND SET

Pounds, Judson Powder ——* Pounds, 40% Giant ——†

Loosened up 58,000 cu. yd. Swell, 1,300 cu. yd. Material broken small

FIG. 26.

north of Andrade was raised above flood heights, the quarry lines were thrown over, and a track was laid on the top of the concrete head-gate, stringers being laid on the cellular pier walls. This track was connected with that on the levee, so that a train could be loaded at the quarry and run over the concrete head-gate and down the levee system, using the main line of the Inter-California Railroad only

for that portion lying on the Hind-Clarke Dam. In the fall of 1907 this railroad was reconstructed and repaired from Calexico to Cocopah, and extended through to the end of the spur levee. The track from this point to Hanlon's Junction was taken over by the Southern Pacific Company, in accordance with arrangements made when the branch line was built. For this, consisting of 61 miles of main line and 21 miles of siding, the Inter-California paid the C. D. Co. approximately $65 000. When this was done the branch line was no longer available for use by the irrigation company, but the right to use the east track or siding on the Hind-Clarke Dam was retained, and the Inter-California was specifically released from any obligation to maintain the dam.

It certainly is not often that a diversion head-gate carries a main-line, standard-gauge railroad, and the fact that this does will give some idea of its size. By thus utilizing it, the heaviest carloads of rock from the Andrade quarry can be hauled over the levee system, and be independent of the Inter-California tracks.

The Southern Pacific Not Reimbursed by the United States Government.

Early in 1907, a bill* was introduced before Congress:

"To reimburse the Southern Pacific Company the amounts expended by it from December first, nineteen hundred and six, to November thirtieth, nineteen hundred and seven, in closing and controlling the break in the Colorado River.

"Be it enacted by the Senate and House of Representatives of the United States of America in Congress assembled, That the sum of one million six hundred and sixty-three thousand one hundred and thirty-six dollars and forty cents is hereby appropriated, out of any money in the Treasury not otherwise appropriated, to reimburse and pay the Southern Pacific Company the amounts paid by it from December first, nineteen hundred and six, to November thirtieth, nineteen hundred and seven, in closing and controlling the break in the Colorado River and thereby saving the overflow and destruction of the Imperial Valley in southern California."

This bill was referred to the Committee on Claims, of which Hon. James M. Miller, of Kansas, was Chairman, and consideration was begun on February 24th, 1908. At this hearing Mr. Maxwell Evarts,

* H. R. 13 997, Sixtieth Congress, First Session.

Counsel of the Harriman Lines, Mr. P. G. Williams, of the Accounting Department, who had been in charge of the accounts of the Epes Randolph, Agent, fund, and the writer, appeared to make explanations and present for examination vouchers covering all items of expense. President Roosevelt was frank and open in urging the justice of the claim, but, of course, was not advised as to whether such amount of money had been expended. Everyone else, however, seemed to be very much afraid of the matter. The Committee on Claims desired information as to the character of the work done and the fairness of the charges therefor, and asked the Secretary of the Interior, James R. Garfield, to have the Reclamation Service Engineers investigate and make a report. The reply was that no funds were available for that particular kind of work. Accordingly, President Roosevelt asked Secretary Garfield, F. H. Newell, M. Am. Soc. C. E., Director of the Reclamation Service, and Mr. Walcott, formerly head of that Service, to take up the matter and report on what the Government, as a matter of moral and equitable obligation, should pay for the service rendered. President Roosevelt at the time, March 11th, in a letter to the Chairman of the Committee on Claims, advised him that this had been done; recalled the dire need for immediate action at the time; stated that negotiations with Mexico were then under way for future action in concert between the two nations; that he did not think the Southern Pacific people should be obliged to wait for the conclusion of these negotiations between Mexico and the United States with regard to future action, but that a rough estimate should be made as to what the United States should pay as reimbursement to the Southern Pacific Railroad, and that it was an act of justice to deal generously in the matter, for the railroad, by its prompt and effective work, rendered to a threatened community a notable service which could in no other way have been done. To facilitate matters, Mr. Evarts agreed with the Committee that Mr. Grunsky, who had been Consulting Engineer of the Secretary of the Interior on all Reclamation Service matters until a few months previous, and who, of all Government engineers, knew most about the Lower Colorado River and the operations of the Southern Pacific Company, should be engaged by the Committee to report on the work done and the expenditures, and that Mr. Evarts would advance the expenses of such investigation and report.

Mr. Grunsky, with the assistance of the American Audit Company, of New York City, investigated the accounts, and reported on April 1st that the structures along the river were adequate and efficient, and fulfilled their intended purposes; that additional protective work would have to be done in the near future; that charges incurred prior to December 7th, 1906, and subsequent to July 21st, 1907, were not properly chargeable to work done as a result of correspondence between the President and Mr. Harriman; that the quantities of material, such as rock, gravel, and earth, covered by the expenditures could not then be ascertained by measurement of completed structures with any degree of precision; that the records presented by the company showing the quantity of rock, gravel, and clay put into the work

TABLE 18.—EXPENDITURES ON THE COLORADO RIVER WORK SUBSEQUENT TO DECEMBER 1ST, 1906.

Labor	$275 310.12
Materials and supplies	261 969.04
Fuel	33 339.58
Freight charges on supplies and materials	613 150.84
Freight fuel	19 073.00
Transportation	12 395.23
Work-train service	7 627.17
Rental of equipment	70 507.54
Commissary supplies and labor	8 356.27
Trackage	31 981.00
Construction of additional levees (contract)	255 378.55
Officers' and clerks' salaries	8 449.08
Office expenses	1 279.20
Traveling expenses	1 437.65
Sundry expenses	1 091.39
Duties	34 717.45
Total	$1 636 063.11

from day to day from January 27th to July 18th, 1907, during which time practically all the work of closing the second break and completing the protection work was done, constituted a reasonable check on the bills rendered, and that the number of carloads of material handled prior to January 27th, 1907, was no doubt correctly reported; that a fair basis for all charges, everything considered, would be cost plus the usual 10% for superintendence, tools, etc.; that this basis had been followed in the accounts, with the exception of the freight, which had been billed at tariff rates and really should have been made at 0.5 cent per ton per mile; that the accounts revised and corrected according to the foregoing showed the total net expenditures to be $1 083 673.97, exclusive of interest.

Mr. Grunsky's report was clean-cut and fair, the inclusion of freight charges at tariff rates being done against the writer's advice.

The Committee on Claims, therefore, now had definite ideas as to the work and cost. The report to President Roosevelt from Messrs. Garfield, Newell, and Walcott (dated March 17th), and mentioned in his letter of March 11th, as to the fair proportion to be repaid the railroad, was forwarded to the Committee, and, after reviewing the situation, including the exchange of telegrams between Mr. Harriman and President Roosevelt on December 19th and 20th, 1906, states:

"Under the circumstances, we do not feel justified in attempting even in a rough way to approximate the burden, other than to state that the principal beneficiaries are six in number: (1) The settlers in the Imperial Valley; (2) the Southern Pacific Co.; (3) the California Development Co.; (4) the Mexican Corporation; (5) the Republic of Mexico, and (6) the United States. Not considering the settlers in the valley, we have five distinct entities among whom the burden might be distributed more or less equally. Thus, a rough estimate might apportion to the United States 20 per cent. of the money expended to reimburse the Southern Pacific Company for the actual expenditures of repairing the break in Mexico. Such proportion would fully comply with your suggestion that the United States Government should act generously toward the Southern Pacific Company, for by prompt and effective work it rendered a notable service to the threatened community of settlers in the Imperial Valley, quite regardless of the ultimate benefit of such action to the railroad company itself."

This recommendation has always seemed remarkable to the writer. The land interests in the Imperial Valley on both sides of the Boundary Line represent fully two-thirds of the present property values which had been threatened; the Southern Pacific Company about one-sixth; the United States, through its Laguna Weir, about one-sixth; the C. D. Co. and the Mexican Co. nothing, because they were both bankrupt, and the Republic of Mexico practically nothing because its interests conserved were wholly prospective, as well as those of the United States as far as irrigable land farther up the river is concerned. Furthermore, it was known that $950 000 had already been subscribed by the people and corporations in Imperial Valley when the President called on Mr. Harriman to start work, as he did and at once stopped such subscription. It is obvious that the railroad could by that time in no possible way collect anything from any other source than the United States for such work, so that such payment as might be made by the United States will represent the grand total reimbursement. Just why this Committee eliminated the land owners

and then considered the remaining five entities as being equally concerned has always seemed remarkable, especially because, when two years later another call for help from Imperial Valley came to President Taft, Congress at once appropriated $1 000 000 to protect the land owners primarily and almost exclusively.

The Committee took no action in the matter until Congress adjourned, and two years later, at the next session, another bill was presented, this time in the Senate (Senate 417, Sixty-first Congress). The matter was gone over again, and the bill with the amount cut to $773 647.25, or 71.4% of that reported by Mr. Grunsky, passed the Senate. The House Committee on Claims made a favorable report. Five members of the Committee submitted a minority report on January 28th, 1911, stating that they did not think there was any legal, equitable, or moral obligation on the part of the Government to pay the railroad company any amount whatever for closing the break; that expenditures were made neither at the request of the Government nor for its benefit; and that the appropriation of such sum would be "Purely a gratuity, a gift of the people's money to the Southern Pacific Railroad Company * * *.

"We oppose this proposed gift to the Southern Pacific Railroad Co. as well as all other gratuities to private enterprise."

The bill did not pass the House.

Damages Caused by the Runaway River.

The first damage caused by the diversion of the river was the flooding of the salt beds and the gradual burying of the entire plant of the New Liverpool Salt Company in the bed of the Salton Sea. The property was probably worth about $125 000.

As the waters continued to rise, they began to threaten the main line of the Southern Pacific Company throughout the basin, and in July, 1905, they reached the rails for a considerable distance. Shooflying was begun and continued from time to time, nothing very aggressive being attempted because of optimistic advices as to when the river would be under control.

Shooflies Nos. 1 to 7, inclusive, were at an elevation of —250 ft. or more. Shoofly No. 11 is 39 miles long, and follows the —200-ft. contour, being determined on when the probabilities of controlling the river before the summer flood of 1906 seemed to be rapidly decreasing.

It was built in February and March, 1906, and was located with a view to being safe from the rising waters for at least two years, the estimates being based on the discharge records of the river at the time of building. As a matter of fact, some sections of the track a few feet below the —200-ft. contour were in trouble in the latter part of October, the water then being 47.5 ft. higher than when the line was surveyed. When the second break occurred, Shoofly No. 12, 48.9 miles long and following the —100-ft. contour, was hurriedly surveyed and graded during January and February, 1907, the outside drainage work being completed on April 1st. Track material for this had been gathered at each end—Mecca and Imperial Junction—during September and October, 1906, when Shoofly No. 11 was threatened. Practically none of the bridging was put in on this latter work, and when only about 4 miles of track were laid it seemed that the river control work would prove effectual, and work was consequently ordered stopped.

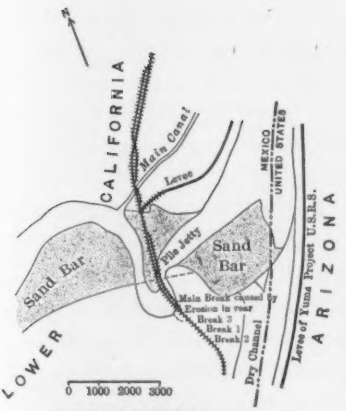
The railroad company also suffered damage along the branch line between Imperial Junction and Calexico. At the crossing of the Alamo River north of Brawley the track was moved five times, the present alignment constituting a shoofly 2 706 ft. long, and introducing 105° of curvature, as compared with 16° 40′ originally. A few miles south, the enlargement of the New River channel made it necessary to construct three shooflies, the last one being 9 086 ft. long and containing 121° of curvature, as compared with 11° in the original alignment.

The total expenses incurred along the Salton Sea, exclusive of the cost of grading Shoofly No. 12, were as follows:

Year.	Labor.	Material.	Totals.	
1905	....	....	$148 183.71	
1906	$181 300.37	$307 763.58	489 063.95	
1907	49 875.96	37 678.08	87 554.04	
				$724 801.70

The damages sustained on the Imperial Branch were as follows:

Year.	Labor.	Material.	Totals.	
1905	....	....	$5 914.01	
1906	$19 222.37	$9 986.53	29 208.90	
1907	2 597.13	142.76	2 739.89	
				$37 862.80

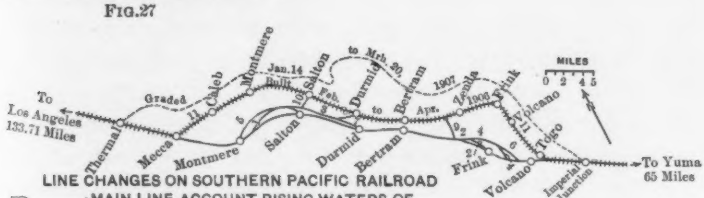


LOCATION OF THE
SECOND BREAK DEC. 7, 1906 AND OF
SECOND CLOSING JAN. AND FEB. 1907

FIG. 27

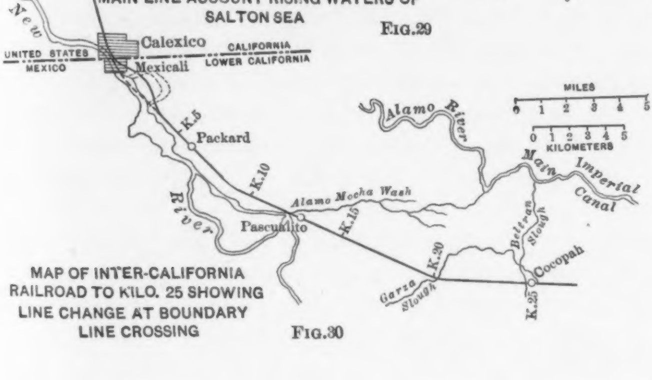
FIVE SHOOFLYS ON THE IMPERIAL BRANCH S.P. CO. AT CROSSING OF ALAMO RIVER AND THREE SHOOFLYS BESIDE THE NEW RIVER BETWEEN ROCKWOOD AND BRAWLEY

FIG. 28



LINE CHANGES ON SOUTHERN PACIFIC RAILROAD MAIN LINE ACCOUNT RISING WATERS OF SALTON SEA

FIG. 29



MAP OF INTER-CALIFORNIA RAILROAD TO KILO. 25 SHOWING LINE CHANGE AT BOUNDARY LINE CROSSING

FIG. 30

The Inter-California Railroad, beginning at the International Boundary Line, was damaged more or less seriously for about 10 miles, the details being as follows:

Entirely rebuilding the road through Calexico and Mexicali and beyond, being cost less value of material recovered	$ 82 822.49	
Repairs to remainder of first 10 km., including water supply	4 259.76	
Repairs from 10-km. point to 14-km. point	21 163.73	
		$108 245.98

Thus the damage sustained by the Southern Pacific on permanent way alone, and not including interruptions to traffic, or expenses of any kind incurred by delayed trains, etc., was $762 664.50, and by the Inter-California $108 245.98, making a total of $870 910.48.

In addition to such damages, the trains of rock hauled during the 3 weeks of the first closing and more than 2 weeks of the second closing, or more than 5 weeks in all, were given rights of way over all except passenger trains, and the slower of these were very frequently delayed in order to hurry material to the front. The demand on the equipment of the road was tremendous, particularly during the second closing, when there were in rock-train service, 1 000 flatcars, 300 "battleships," 4 steam shovels, 10 work trains, exclusive of rock trains from the quarries other than at Andrade, etc. Indeed, for about 10 days, practically no freight was hauled out of the Port of San Pedro because of lack of equipment. The degree of the strain is shown by a telegram the writer received from the Superintendent of the Los Angeles Division, just before the second closing was accomplished, asking information as far in advance as possible "when work will slow up because I want to make arrangements to resume operating the division."

About 3 000 acres of cultivated land and 10 000 acres of uncultivated and public lands were practically destroyed and rendered unavailable for agricultural purposes under existing conditions, the total value of which depends on whether the present or prospective worth is considered. Perhaps a very fair figure would be $50 per acre for cultivated land and $10 per acre for raw lands, making the total damage to the land about $250 000.

Various individual settlers in Mexico and the United States suffered more or less severe injury from inundation of crops, etc., as distinct from land damages, and these probably amounted to not more than \$150 000. A number of claims for damages sustained on the American side of the line were combined in a suit totaling \$490 000, but this included damaged lands estimated at prospective rather than real and present figures.

The most serious injury done to settlers was the entire stoppage of the water supply in the canals of Districts Nos. 6 and 8 from the summer of 1906 until January, 1907. No claim was ever presented for these damages, and nothing more than crude guesses can be made as to the amount. The result practically forced the depopulation of more than 30 000 acres of land, of which about 12 000 acres were under cultivation. The effect on the development of the valley at the time was not very great, but fear of repetition of a break, as much as anything else, has, until recently, retarded the region to a considerable extent.

Because of the tremendous expenditure involved in re-diverting and holding the river in its course, up to December 5th, 1907, when the Southern Pacific advanced funds for the work directly instead of to the C. D. Co., the original \$200 000 loan to the C. D. Co. had grown to \$1 100 000, and this was swelled by later bills, interests, etc., to approximately \$1 375 000, by January 1st, 1909. In addition to owing this large sum for cash actually advanced, and for which payment could not be disputed successfully, a judgment for \$458 246.23, in favor of the New Liverpool Salt Company had been rendered in the United States District Court toward the end of November, 1907, and there were claims from the Southern Pacific Company and others aggregating \$1 360 000, two-thirds of which were from the latter company. The runaway river rendered hopelessly bankrupt the C. D. Co. and the Mexican Co.

Benefits.

To almost every cloud there is a silver lining, and this is no exception. It is now known that the diversion would have occurred very soon. The event showed the existence and nature of the danger and the necessity for guarding against it. Much more important was the development and standardizing of methods of closing future crevasses which might occur. Incidentally, the information of this character afforded to the Engineering Profession in general will doubtless prove of much

value, though this cannot be considered as a benefit to the region in question.

By far the greatest benefit was the erosion of the great Alamo and New River barrancas and the creation of the main features of a complete and comprehensive drainage system for the entire Imperial Valley. The natural slope of the ground is remarkably uniform, with a grade of about 5 ft. per mile, and the very small, shallow channels of the New and Alamo Rivers were the only rudiments of satisfactory drainage, from an irrigation point of view. The Salton Sink is the natural drainage sump for the region, and its absolute control should have been acquired in the very beginning, either by the irrigating company or by the land holders of the valley. In the litigation which followed the destruction of the salt works, the New Liverpool Salt Company, as owner of the submerged land, obtained a decree perpetually enjoining and restraining the C. D. Co. from diverting water from the Colorado River in excess of the substantial needs of the people dependent on the canal for water supply for domestic, irrigation, and such other lawful purposes as the same may be applied to, and with a further provision as to the control of the water diverted so that it will not overflow on the lands of the complainant. Later proceedings resulted in a most remarkable construction of the last portion of the injunction, so that now the Salt Company practically cannot object to the use of the basin as a natural sump. Judgment has just (October, 1912) been rendered for a total of $78 000.

Providing for the region an efficient drainage system to carry all the waters into the Salton Sink would have required a large amount of money—so large that the date of its establishment would have been delayed very far into the future, much too far for the valley's real interests. This is true because it is plainly not the business of an irrigation company to supply a drainage system, and all other interests of the valley are very much divided because of the mutual water company plan of organization, and because of the usual lack of co-operation among farmers. Furthermore, the need for drainage of irrigated land is usually not recognized in time, and not admitted when it is recognized. Indeed, in spite of the rather alkaline character of the lands in Imperial Valley, as already explained, it was not until November 1st, 1911, that any serious suggestion was made for a community drainage

canal—in Imperial Water Company No. 8—the reasons then chiefly urged being:

"The loss of ground and bad appearances caused by the ends of the irrigated lands being covered with weeds or wild grass or perhaps nothing at all as the result of standing water."

A few spots in Imperial Valley are beginning to indicate an undesirable increase in alkalinity, and it is most fortunate that the magnificent main drainage ways of the Alamo and New River channels exist.

TABLE 19.—ANALYSIS OF WATER OF NEW RIVER.

Sample taken at Brawley, June 6th, 1908, and submitted by Mr. F. W. Roeding, Irrigation Investigations, Berkeley, Cal.

		Grains per gallon.	Parts per 1 000 000.
Alkali	Potassium Sulphate very small, and Sodium Sulphate (Glauber's salts), etc.	12.87	221
Alkali	Sodium Chloride (common salt)	389.40	6 675
Alkali	Sodium Carbonate (sal soda)	0.99	17
Alkali	Calcium Chloride	14.91	256
Alkali	Magnesium Chloride	37.26	639
	Calcium and Magnesium Carbonates, etc., large Calcium Sulphate (gypsum) chiefly	94.29	1 617
	Silica	1.22	21
	Organic matter chars, and chemically combined water	57.44	985
	Total	608.38	10 431

SALTON SEA.

From a geological and spectacular point of view, the creation of the Salton Sea in so short a time was one of the most striking effects of the river diversion. The water filled the basin to a maximum elevation of —197.4 U. S. G. S. datum, or —204.2 S. P. datum, the maximum depth of water being 76 ft. The total area covered at this time was about 445 sq. miles, with a length of 50 miles and a width of from 10 to 15 miles. With the exception of the Great Salt Lake and Lake Michigan, the sea was the largest body of water lying wholly within the United States.

The water rose at the maximum rate during the latter part of June, 1906, when it gained nearly 7 in. per day, or 15.4 ft. during that month. From the reconnaissance map of the Salton Sink, pub-

lished by the U. S. Geological Survey in 1905, it would appear that the areas of the various contours of the basin are about as follows:

Elevations.	Square Miles.
—280	0
—250	240
—200	445
—150	650
—100	920
— 50	1150
— 0	1750
+ 30 (old beach line)	1950

TABLE 20.—ANALYSES OF WATER FROM THE SALTON SEA AND THE COLORADO RIVER AT VARIOUS SEASONS.

As made by Dr. W. H. Ross, of the Agricultural Experiment Station, University of Arizona, Tucson, Ariz.

	Salton water, surface of deepest portion, June 3d, 1907.	Colorado River water, low winter stage, January 10th–March 26th, 1900.	Colorado River water, rising summer stage, from melting snow, March 27th–April 30th, 1900.	Colorado River water, summer flood from melting snow, May 1st–June 29th, 1900.	Colorado River water, low summer stage, June 30th–August 26th, 1900.	Colorado River water, affected by local floods, August 27th–October 1st, 1900.	Colorado River water, summer floods from Arizona, October 2d–November 19th, 1900.	Colorado River water, low winter stage, November 20th–January 24th, 1901.
Total soluble solids at 110° Cent. Parts in 100 000..	364.8	92.9	67.4	32.2	36.1	71.4	104.5	87.1

COMPLETE ANALYSIS OF SOLUBLE SALTS, STATED BY IONS. PARTS IN 100 000.

Sodium, Na	111.1	19.0	15.3	5.5	7.6	14.6	18.2	16.0
Potassium, K	2.3	1.1	2.1	1.0	1.3	1.8	2.1	1.2
Calcium, Ca	9.9	7.5	4.9	4.2	4.6	7.7	12.5	9.2
Magnesium, Mg	6.4	3.1	2.0	1.2	1.2	2.2	2.8	2.8
Aluminum, Al	0.031	....	....	....	....	....	....	....
Iron, Fe	0.005	....	....	....	....	....	....	....
Chlorine, Cl	169.7	20.5	13.9	4.5	6.9	15.8	17.5	18.1
Sulphuric, SO_4	47.6	26.1	19.4	7.2	7.6	19.7	35.6	23.8
Carbonic, CO_3	6.6	7.4	8.4	7.2	7.7	10.0	12.2	10.7
Silicic, SiO_3	1.2	4.6	2.1	2.1	3.3	2.3	2.2	2.1
Phosphoric, PO_4	0.018	....	....	....	....	....	....	....

Practically all the water which enters Salton Sea comes from the Alamo and New Rivers, which, under normal conditions, are now important chiefly as drainage channels for the Imperial Valley. Frequently, however, very heavy precipitation occurs in violent storms over small portions of the area draining into the basin, but the run-off, though occasionally of considerable quantity, is not relatively important. The total annual inflow is at present probably 200 000 acre-ft., or sufficient to cover the surface of the sea about 0.7 ft. in depth, while the evaporation is probably about 6 ft. and the percolation insignificant.

TABLE 21.—COMPOSITION OF OCEAN WATER.

(This table gives the mean of 77 analyses made by the Challenger Expedition, Challenger Report, Physics and Chemistry, Vol. 1, 1884, p. 203.) Stated by Ions. Parts per 100 000.

Sodium (Na)	1 071
Potassium (K)	39
Calcium (Ca)	42
Magnesium (Mg)	130
Sulphate (SO_4)	270
Chloride (Cl)	1 935
Bromide (Br)	6
Carbonate (CO_3)	7
	3 500

Quite a little speculation has been indulged in regarding the length of time which would have been required to fill the Salton Sea had the Colorado River not been re-diverted. Most of such computations are based on too low an average flow of the river past Yuma, which it now seems is in excess of 12 000 000 acre-ft. per annum. As a matter of fact, however, the inflow from the Imperial Valley region will constantly increase, and the quantity evaporated will decrease directly with the decrease in water surface exposed, so that a balance will be reached probably in such time as the inflow will average between 350 000 and 500 000 acre-ft. per annum from all sources, and the exposed surface will cover between 60 000 and 80 000 acres. At such time the maximum depth of the sea will be between 8 and 10 ft.

The sea has already (January, 1912) fallen about 22 ft., and has exposed approximately 115 sq. miles which were under water. The salt beds were dissolved to such an extent as to render the water of the sea quite salt, unfit entirely for drinking purposes, and it was assumed that the land which it covered would be hopelessly alkaline.

This does not seem to be the case, and a very considerable acreage of such exposed land is being cultivated with entire success.

Much speculation was indulged in regarding the effect of this body of water on the rainfall and climate of the Southwest. A careless consideration of the precipitation on the drainage area of the river, particularly that of the Gila water-shed, before and after January, 1905, might lead to the conclusion that the effect is quite marked. The period from January 1st, 1905, to date has been one of very heavy rainfall throughout the Southwest, its most remarkable part being in the early part of January, 1905, which was before the formation of the Salton Sea. Professor Alfred J. Henry* points out the fallacy of such an opinion, as follows:

"Admitting, for the sake of argument, that a body of water * * * existed * * * 60 miles long, 8 miles broad, and say 25 feet deep on the average. * * * The cubic contents would therefore be 60 x 8 x 0.0047 = 2.2 cubic miles of water. The normal annual rainfall of Arizona * * * is 11.75 inches. [while in 1905 it was 26.6 inches], or an excess of 14.85 inches, an amount more than equal to the normal annual rainfall. * * * As the area of the Territory is 113,956 square miles, * * * the number of cubic miles of rain that fell in Arizona in excess of the average was * * * 27, * * * twelve times greater than the total volume of the Salton Sea. In other words, the total volume of the latter would barely suffice to produce one-twelfth of the surplus rain that fell in Arizona, to say nothing of the rainfall in adjoining regions. The total amount of water now in Salton Sea, if uniformly distributed in Arizona, would cover the Territory to the depth of about an inch and a quarter, or the equivalent of one good soaking rain."

As a matter of fact, the area of the Salton Sea and Laguna Maquata combined are insignificant when compared with that of the Gulf of California, and are just about as far from Arizona. Professor Henry concludes that the Salton Sea has increased the relative humidity in the immediate vicinity in a slight measure; that it is improbable that any considerable portion of the vapor it gives off passes beyond the immediate confines of the desert; and that there might be a tendency toward lower maximum and higher minimum temperatures in a narrow zone immediately surrounding the sea, particularly on the leeward side.

* "The Salton Sea and the Rainfall of the Southwest," *Monthly Weather Review*, Washington, December, 1906.

INTERNATIONAL NEGOTIATIONS.

When the Mexican Co. obtained its concession from the Mexican Government, Col. Jacobo Blanco, then Chief of the International Boundary Line Commission for the Mexican Government, with headquarters at El Paso, was appointed Inspector of the Mexican Co. and its operations. In 1906, Col. Blanco died, and his successor on the International Boundary Line Commission was Señor Fernando Beltran

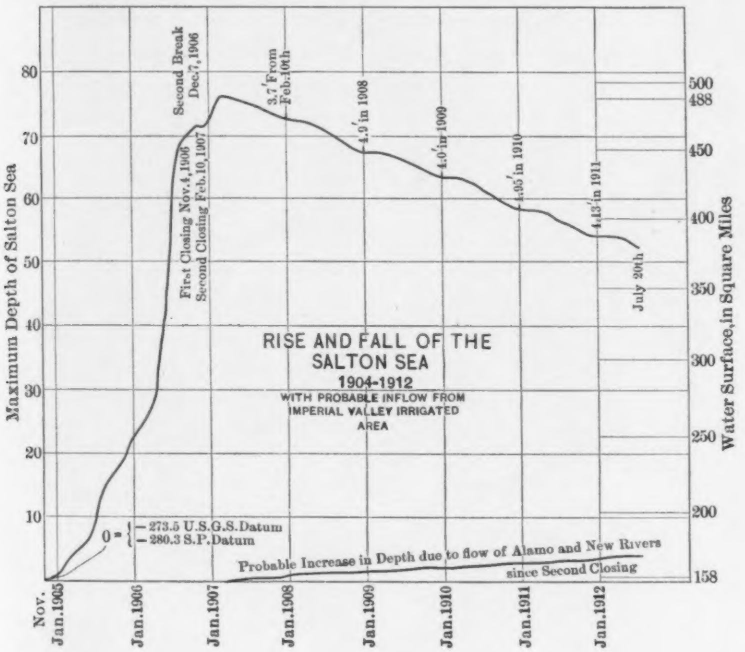
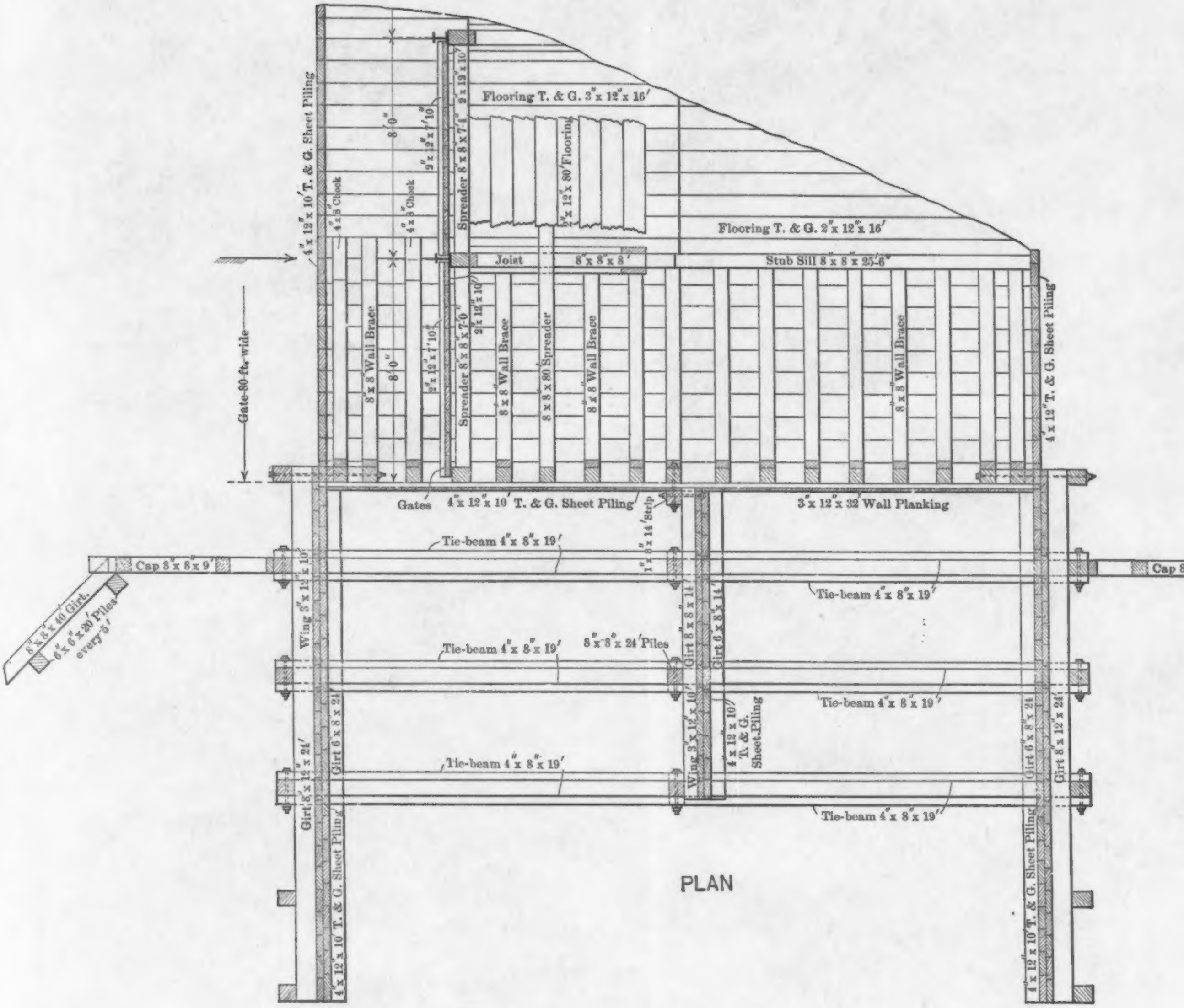


FIG. 31.

y Puga, who was also appointed his successor as Inspector of the Mexican Co. The writer considers this appointment an exceedingly fortunate one, as Señor Beltran y Puga is an exceptionally efficient, aggressive, and fair-minded man, and an engineer, with whom it has always been a satisfaction to transact business. Immediately on his appointment he acquainted himself with the conditions along the river and with the affairs of the Mexican Co., and has always acted promptly and with decision.

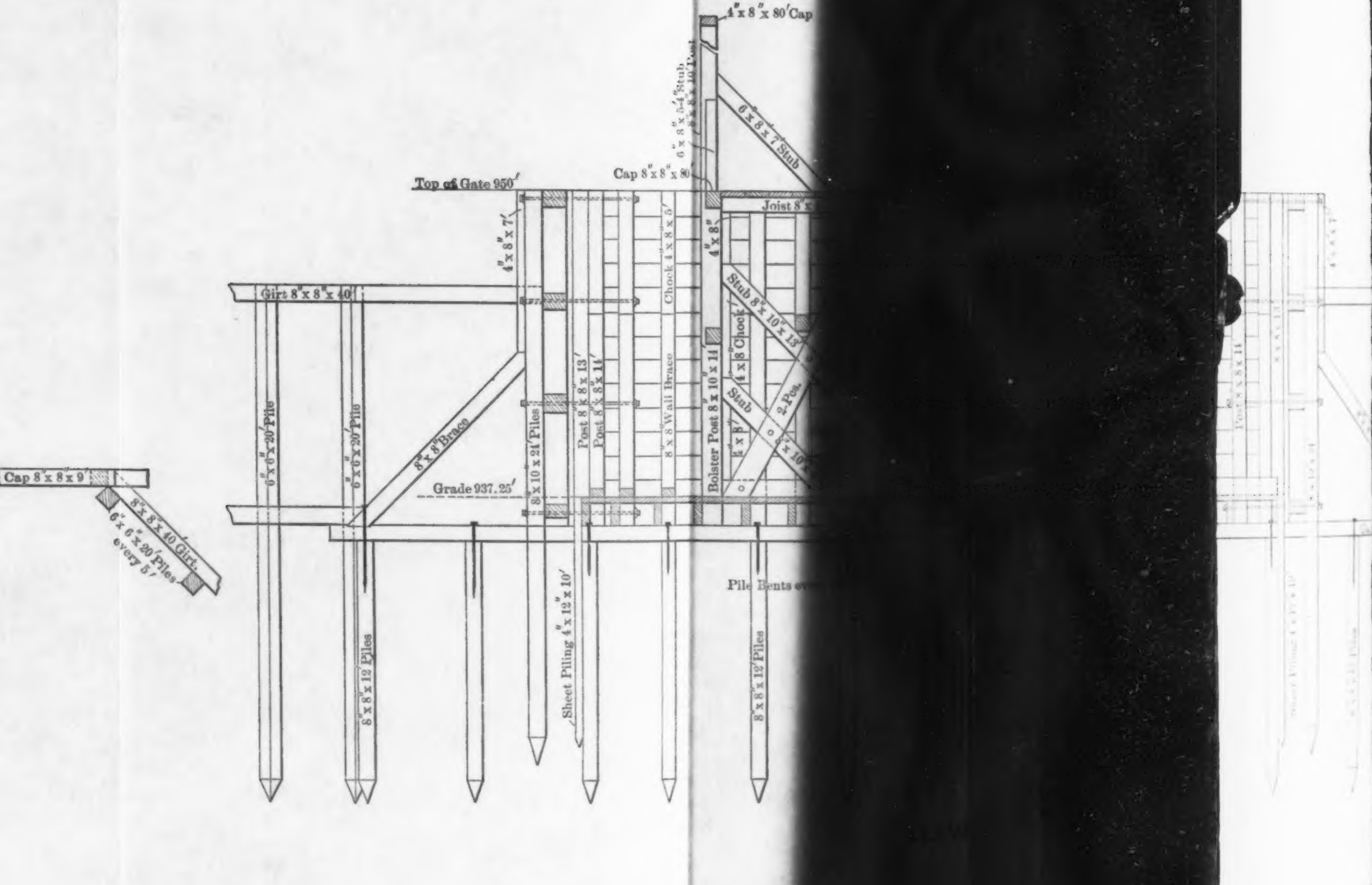
In the spring of 1908 the United States State Department appointed Mr. Louis C. Hill, Supervising Engineer of the U. S. Reclamation Service, to represent the United States on a joint commission to work out the provisions of a treaty with Mexico for the control of the Lower Colorado River and an equitable distribution of its waters. This appointment was in line with President Roosevelt's promise to Mr. Harriman, and was doubtless in a measure brought about at this particular time by the failure of Congress to take any action on the bill to reimburse the Harriman interests. At the request of the United States, the State Department of Mexico appointed a Commissioner, and rather naturally selected Señor Beltran y Puga to represent the Mexican Government, this gentleman's appointment being made on May 7th, 1908, and practically simultaneous with the appointment of Mr. Hill. Both gentlemen were instructed to act together and make a study of the works and operations necessary to complete international control of the lower Colorado River and render impossible a repetition of the recent disaster and the complete utilization of the waters of the river, such study to be in whatever detail might be deemed necessary.

This Commission never had a formal meeting, which is very much to be regretted, considering the importance of the matter. Very shortly after their appointment, the Commissioners had an informal meeting at which, according to private conversations which the writer had with both gentlemen, it appears that Señor Puga submitted, in the form of a written memorandum as the basis for discussion, the suggestion that both Governments cancel the existing treaties regarding the navigability of the river; that regulation of the flow of the river by extensive storage works in the upper portions of the drainage basin was desirable; that both Governments determine the priority and extent of existing water rights and fix rules for granting future water rights; that a joint international commission should make all engineering and other investigations necessary, and divide the costs thereof; that all plans or projects existing or proposed along the river should be submitted to the investigation of the joint commission; that a report be made outlining, in a general way, the work to be done, for the purpose of having a full and complete treaty arranged and the necessary definite appropriations set forth; and that it would be agreeable for Mexico to negotiate a treaty, either preliminary or



Flooring T. & G. 3"x 12"x 16'
2"x 12"x 10'
8'0"
2"x 12"x 10'
Spreader 8 x 8 x 7'4"
2"x 12"x 80' Flooring
4"x 12"x 10' T. & G. Sheet Piling
4"x 8" Chock
4"x 8" Chock
Flooring T. & G. 2"x 12"x 16'
Joist
8"x 8"x 8'
Stub Sill 8"x 8"x 25'-6"
Gate-80-ft. wide
8"x 8" Wall Brace
8'0"
2"x 12"x 7'10"
Spreader 8"x 8"x 7'-0"
2"x 12"x 10'
8"x 8" Wall Brace
8"x 8"x 80' Spreader
8"x 8" Wall Brace
8"x 8" Wall Brace
4"x 12" T. & G. Sheet Piling
Gates
4"x 12"x 10' T. & G. Sheet Piling
Strip
3"x 12"x 32' Wall Planking
Tie-beam 4"x 8"x 19'
Cap 8"x 8"x 9'
Cap 8"
1"x 8"x 14'
Tie-beam 4"x 8"x 19'
Wing 3"x 12"x 10'
8"x 8"x 40' Girt.
6"x 6"x 20' Piles every 5'
Girt 8"x 8"x 14'
Girt 6"x 8"x 14'
Tie-beam 4"x 8"x 19'
8"x 8"x 24' Piles
Tie-beam 4"x 8"x 19'
Wing 3"x 12"x 10'
4"x 12"x 10' T. & G. Sheet Piling
Girt 6"x 8"x 24'
Girt 8"x 12"x 24'
Tie-beam 4"x 8"x 19'
Tie-beam 4"x 8"x 19'
Girt 6"x 8"x 24'
Girt 8"x 12"x 24'
4"x 12"x 10' T. & G. Sheet Piling
4"x 12"x 10' T. & G. Sheet Piling

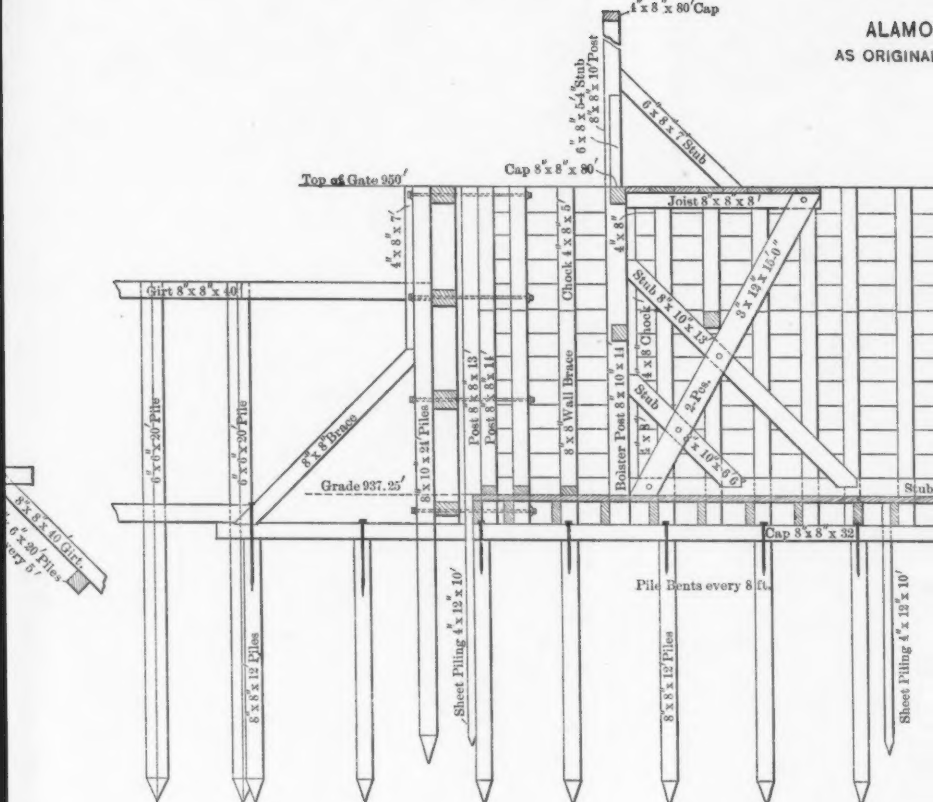
PLAN



4″x 8″x 80′ Cap
6″x 8″x 5-4′ Stub
8″x 8″x 10′ Post
6″x 8″x 7′ Stub
Cap 8″x 8″x 80′
Top of Gate 950′
Joist 8″x
4″x 8″x 7′
4″x 8″
Girt 8″x 8″x 40′
Chock 4″x 8″x 5′
Stub 8″x 10″x 13′
4″x 8″ Chock
Stub
2 Pcs.
Post 8″x 8″x 13′
Post 8″x 8″x 14′
8″x 8″ Wall Brace
Bolster Post 8″x 10″x 14′
6″x 6″x 20′ Pile
8″x 8″ Brace
8″x 10″x 21′ Piles
Cap 8″x 8″x 9′
Grade 937.25′
8″x 8″x 40′ Girt.
6″x 6″x 20′ Piles every 5′
8″x 8″x 12′ Piles
Sheet Piling 4″x 12″x 10′
Pile Bents eve

ALAMO

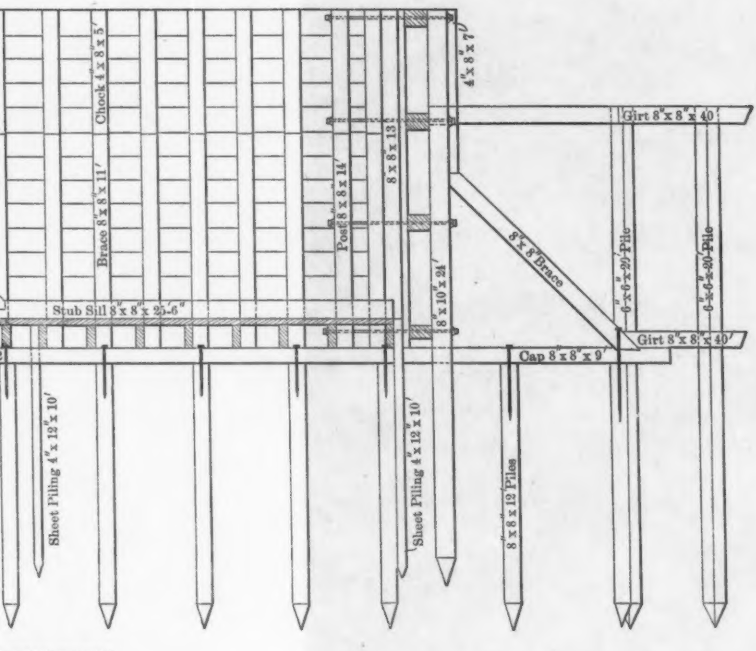
AS ORIGINAL



ELEVATIO

PLATE CXXVI.
PAPERS, AM. SOC. C. E.
NOVEMBER, 1912.
CORY ON
IRRIGATION AND RIVER CONTROL,
COLORADO RIVER DELTA.

ALAMO WASTE-GATE
ORIGINALLY CONSTRUCTED



LEVATION

final, at the earliest possible moment. It seems, however, that Mr. Hill could not find time to attend to the matter, nor were special funds placed at his disposal to defray the necessary expenses. At any rate, after two years of inaction, on May 17th, 1910, the United States Department of State recalled Mr. Hill and substituted Mr. Wilbur Keblinger, Secretary of the American Side of the International Boundary Commission with Mexico. Unfortunately, this change of American commissioners did not have any result. Although Mr. Keblinger lives in El Paso, which point is also Señor Puga's headquarters, the Commission has never had a formal meeting.

It is hoped that discussion on this paper will bring out the reason for not dealing with Mexico's commissioner, appointed at the request of the State Department of the United States, particularly as the writer knows Mr. Hill to be an unusually tactful, courteous, and aggressive gentleman, and an efficient engineer. In any event, there seems to be no doubt that the Mexican Government has been not only willing, but anxious, to arrange for a satisfactory joint control of the lower Colorado River, and that the responsibility for nothing of this sort having been done rests with the United States.

Building of Volcano Lake Levee.

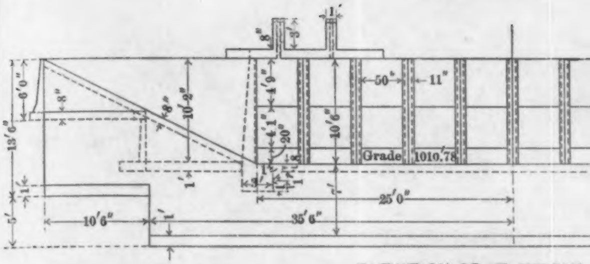
The extraordinary quantity of water which got into Volcano Lake during the summer flood of 1907 raised it higher than it had ever before been known to be, and a large quantity of water passed northward through the New River outlet. Furthermore, a reconnaissance showed that the large quantity of overflow water had started cutting back fingers from the Volcano Lake region toward the river, which indicated the probability of the diversion of the Colorado River below the divide of the delta cone, along the Pescadero, Abejas, or Paredones. Therefore it seemed that another portion of the complete levee system, as originally planned by the writer to hold back the overflow waters of the Colorado from the Salton Basin, should be constructed—that portion from the mountains on the west side of the valley eastward along the north of Volcano Lake to the low-lying divide or ridge farther on. The C. M. Co., however, objected to this, as Volcano Lake is entirely on its land and its utilization for irrigating a portion of that company's lands was considered, but found impracticable because of the great variation in the water surface, the inundation of a

part of the land, however, at flood times was simple, and permitted securing considerable pasturage. The C. M. Co., however, was willing to give the right of way and permit the construction of the protective works provided a permanent head-gate were installed at Cerro Prieto through which water might be let into the New River when the lake was full. This was finally agreed to, and arrangements were completed under which the Southern Pacific Company agreed to pay the Mexican Co. for constructing this gate on condition that the Mexican Co. would arrange to have 8 miles of levee to the east built. This was done and the gate and levee were completed just before the summer flood of 1908 began to throw its waters into Volcano Lake. After one season the Mexican Government compelled the removal of this intake gate and the levee to be built around in front of it, so that it is not now in service. Plans for the gate and levees were presented to the Mexican Government Inspector of the Mexican Co., Señor Puga, and it was understood that their construction was approved and permission verbally given to begin work before filing maps and drawings and having them approved or changed as required by the Departmento de Fomento in such cases—this on account of emergency. It was not so understood by Señor Puga, and the construction of the gate was a needless expense.

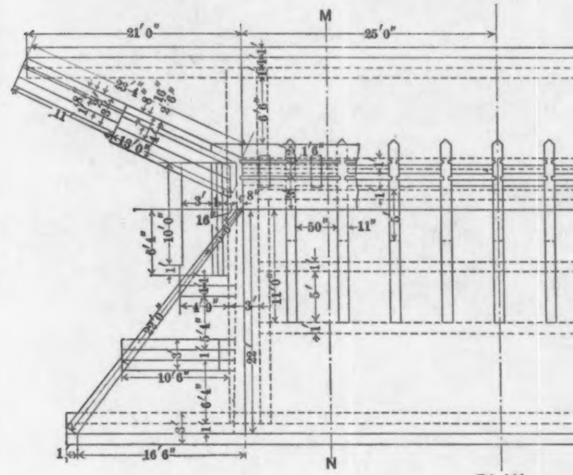
All this work was arranged for and practically done while Congress was considering the payment of the bills for the second closing and subsequent protective work, and when there was no reason to doubt that a fair adjustment would be made.

Reconstruction of Irrigation System.

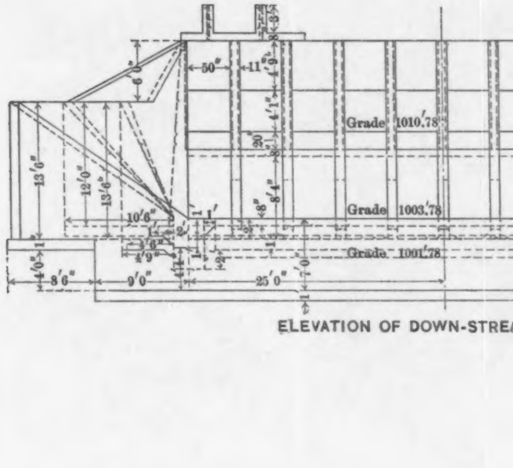
As soon as the river control work was assured, arrangements were made to ascertain the exact condition of the irrigation system of the valley, and what was necessary and advisable to do in connection with it. Accordingly, Mr. F. C. Herrmann, who was added to the engineering staff on February 1st, 1907, was placed in charge of this work. The damage done by the flood was confined almost entirely to carrying away the flume by which the West Side Main Canal crossed New River, and a similar flume, 20 miles north over New River, carrying water from the Central Main to supply Imperial Water Company No. 8. To rebuild the latter was impracticable on account of the immense barranca which had been created at the old



ELEVATION OF UP-STREAM

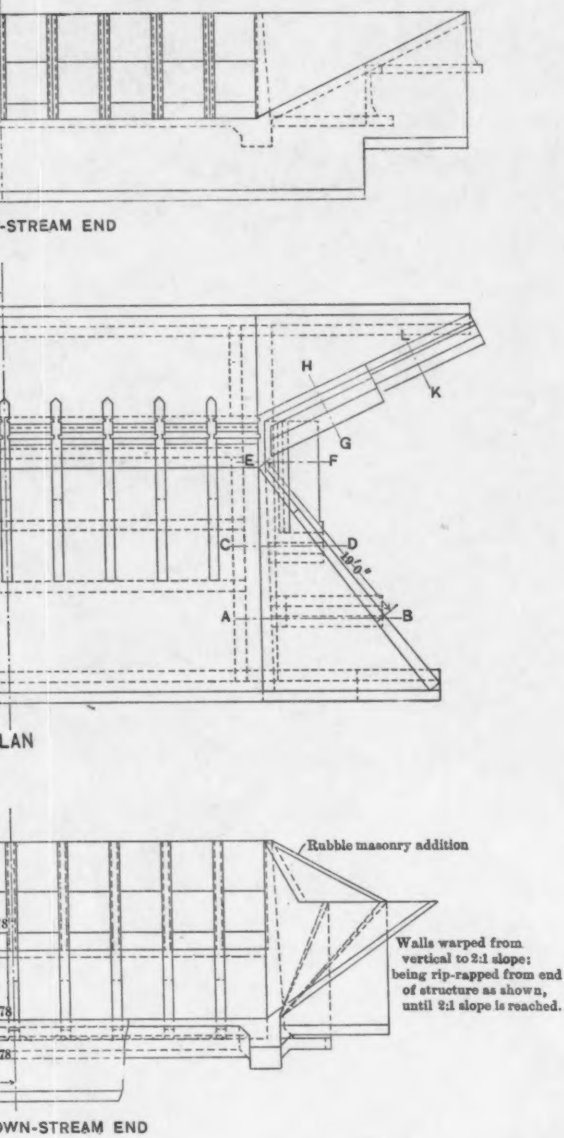


PLAN



ELEVATION OF DOWN-STREA

PLATE CXXVII.
PAPERS, AM. SOC. C. E.
NOVEMBER, 1912.
CORY ON
IRRIGATION AND RIVER CONTROL,
COLORADO RIVER DELTA.



crossing, and it was decided to enlarge the West Side Main and extend it northward so that all the territory west of New River would be served thereby.

A wooden flume, supported by wooden piling, was designed to cross the New River gorge very close to the location of the original flume, carried away about March, 1906. Work was begun on the structure and rushed to completion. This flume is worthy of note because of its height, length, and cost, as a quasi-temporary structure. It is 1 860 ft. long and the maximum height of the trestle is 55 ft. It supports a rectangular flume, 16 ft. wide and 6 ft. deep, built of 2-in. redwood lumber with ship-lap joints. It has given excellent service, and the leakage has been notably slight from the time it was first put into service.

Surveys for the reconstruction, enlargement, and extension of the West Side Main were hurried, and contracts were let for the work, which was well under way when the financial panic of November, 1907, occurred. The contractors were forced into bankruptcy, and the work was completed by their bankruptcy trustee, which caused considerable delay, but water was turned through the reconstructed West Side Main late in December, 1907. This canal is 28 miles long—7 miles in Mexico and 21 in California—and has a capacity varying from 800 to 400 sec-ft., with 760 000 cu. yd. of earthwork moved at a total cost of $86 000, and $5 000 for two temporary structures.

Some little time later the Rose Levee, across the Alamo channel at Holtville, was reconstructed, with a waste-gate to pass the excess of water coming through the Holton Power Company's plant and through the Alamo Waste-gate farther up that channel in Mexico, and a head-gate for the Rositas Canal. Both of these are of reinforced concrete, the waste-gate being of interesting design and capable of passing 2 500 sec-ft. with a total drop of 17 ft. In this way water which must be furnished to the Holton Power Company under its contract is picked up below the plant and utilized for irrigation, as was the original intention when the contract was made. These two permanent structures and the earthen dam cost $55 250.

Another important permanent structure, known as the Seven-Foot Drop, was built in the Central Main just south of the Boundary Line at a cost of $23 760, including three small structures adjacent to it. but not exactly a part thereof. This structure takes the place of a

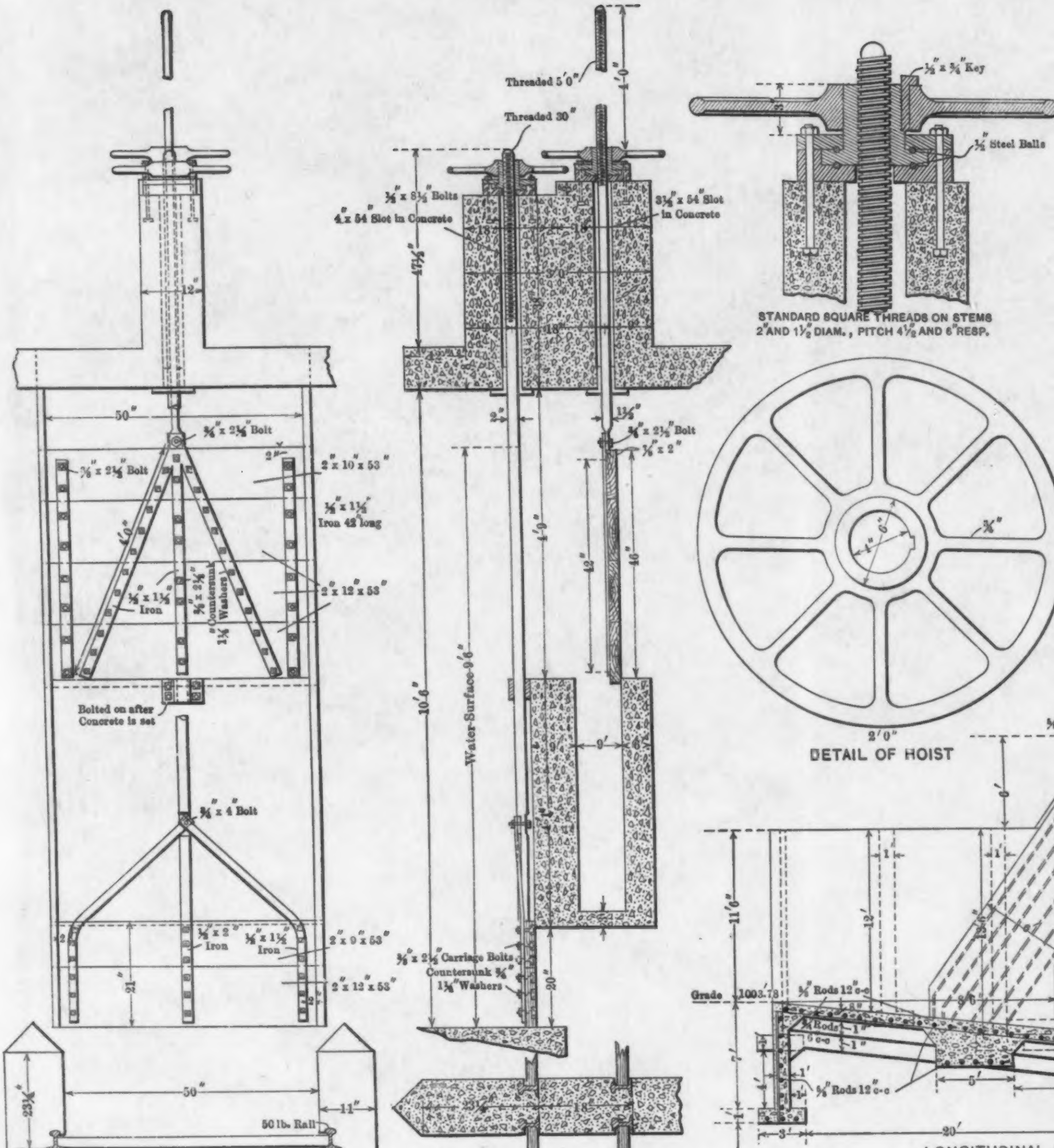
wooden 10-ft. drop nearly 2 miles farther on, which, by the way, had passed considerably more water than it was designed for and was in fairly good condition when removed after 8 years of service.

The chute of the Alamo Waste-gate was repaired and again extended down stream, the result being a quite remarkable construction, Plate CXXVI, which has never shown any signs of weakness against a head of approximately 35 ft.

Surveys and Designs for Extensions.—At the same time, surveying parties were assembled and topography taken, with 1-ft. contour intervals, on 230 sq. miles. This included the west side of New River and a strip in Mexico adjoining the Boundary Line averaging 3 miles wide and running from the West Side Main crossing of the Boundary Line to about 4 miles east of Sharp's Heading and thence generally following the Alamo channel almost to the levee system along the river, the Alamo channel being carefully mapped and cross-sectioned. Much of this work was done in the heat of the summer, five complete surveying parties being on the work until after August 1st, when the force gradually lessened. A large part of the area was practically cleared land, but much of it was covered with a dense undergrowth, which made progress very slow, and, by cutting off the breeze, accentuated the severe climatic conditions. Nevertheless, the cost of this work was about 57½ cents per acre, or $37 per sq. mile. This experience showed that, while the great summer heat there is quite disagreeable, it does not render engineering field work by any means impracticable, and indeed does not increase the cost more than 10 per cent. One agreeable feature of the very hot season is that the temperature is too great for flies, so that they practically disappear.

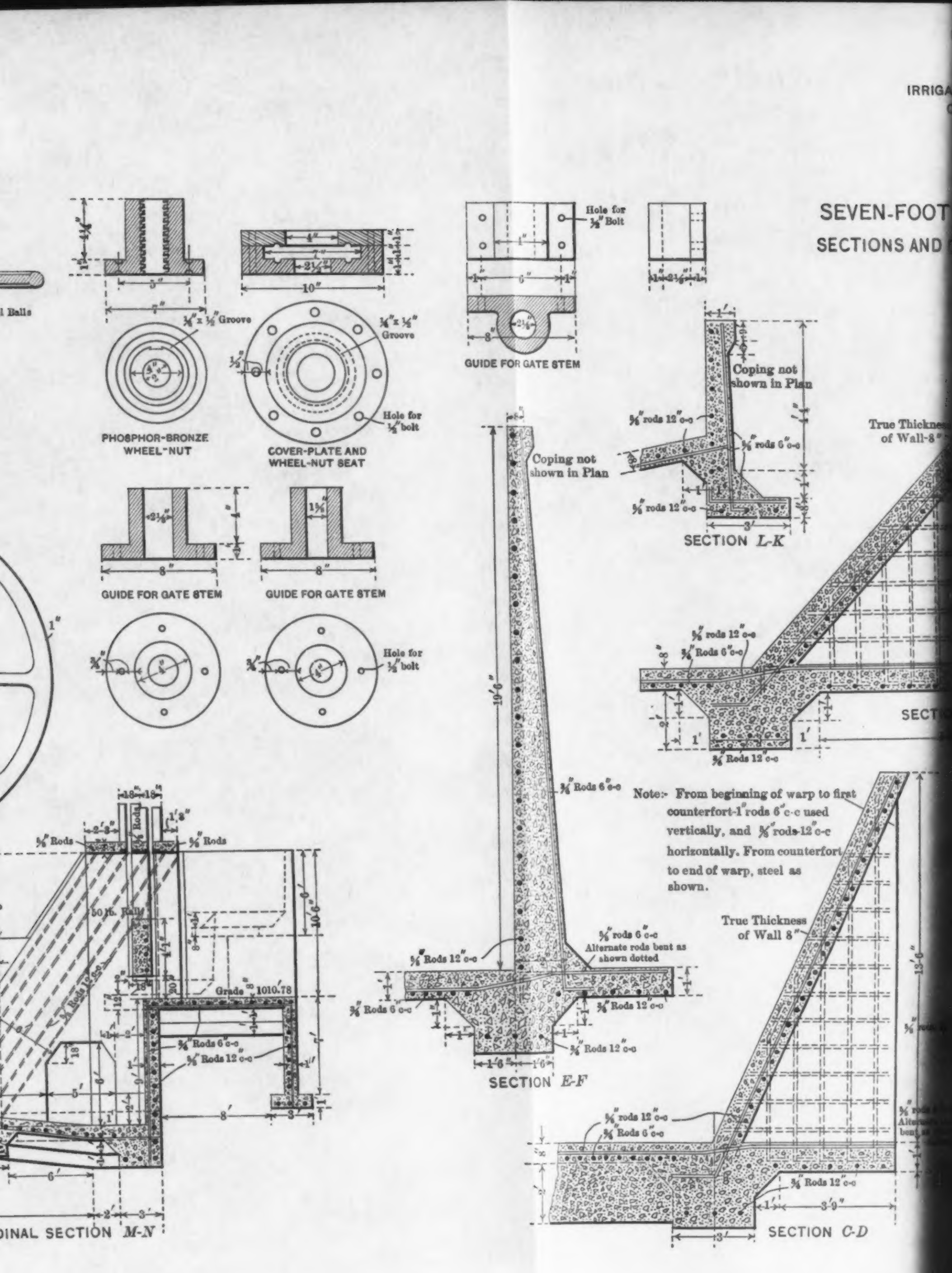
Cross-sections and current-meter observations were taken at various points in the Main Canal and in the important canals of the distribution systems of the various mutual water companies, to determine capacities, losses, etc.

When these data were compiled and put into form, estimates were made for the reconstruction and enlargement of the existing system and for extensions to cover a great deal of new territory. The estimate for this work was approximately $900 000 with temporary structures, and including considerable improvements in the main Alamo channel from the river to the controlling works in the valley; and $2 200 000 with permanent structures at essential points and replacing the Alamo

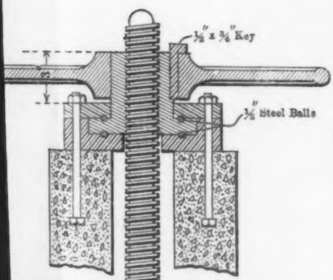


DETAIL OF HOIST

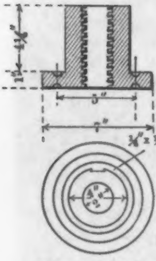
LONGITUDINAL



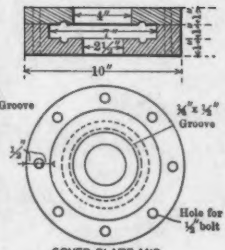
IRRIGA
SEVEN-FOOT
SECTIONS AND
PHOSPHOR-BRONZE WHEEL-NUT
COVER-PLATE AND WHEEL-NUT SEAT
GUIDE FOR GATE STEM
GUIDE FOR GATE STEM
GUIDE FOR GATE STEM
Hole for ½" Bolt
Hole for ½" bolt
Hole for ½" bolt
¼" x ½" Groove
¼" x ½" Groove
Coping not shown in Plan
Coping not shown in Plan
SECTION L-K
SECTION E-F
SECTION C-D
True Thickness of Wall 8"
Note:- From beginning of warp to first counterfort-1" rods 6" c-c used vertically, and ⅜" rods 12" c-c horizontally. From counterfort to end of warp, steel as shown.
⅜" rods 12" c-c
⅜" Rods 6" c-c
⅜" Rods 12" c-c
Alternate rods bent as shown dotted
50 lb. Rails
Grade 1010.78
DINAL SECTION M-N



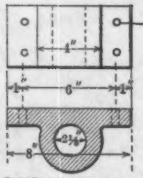
STANDARD SQUARE THREADS ON STEMS
2" AND 1½" DIAM., PITCH 4½" AND 6" RESP.



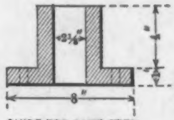
PHOSPHOR-BRONZE
WHEEL-NUT



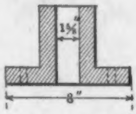
COVER-PLATE AND
WHEEL-NUT SEAT



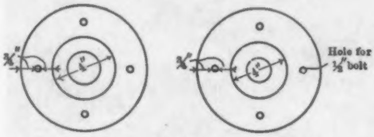
GUIDE FOR GATE STEM

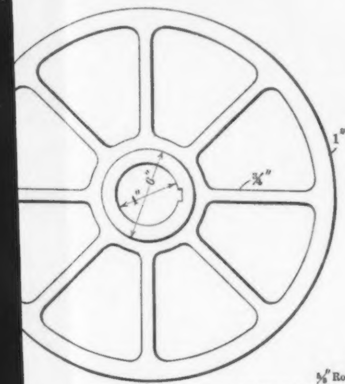


GUIDE FOR GATE STEM

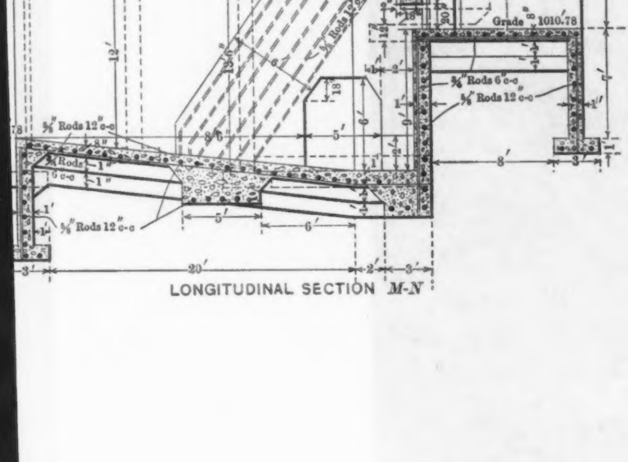


GUIDE FOR GATE STEM

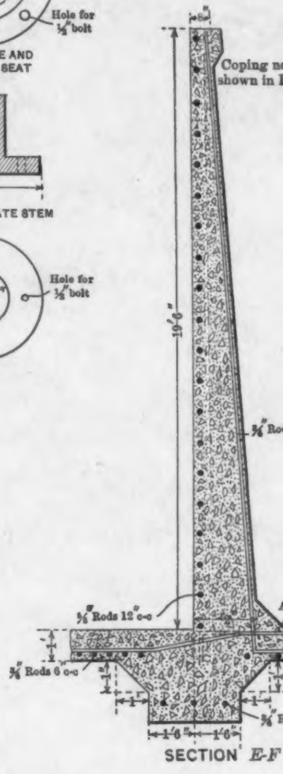




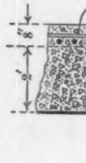
DETAIL OF HOIST



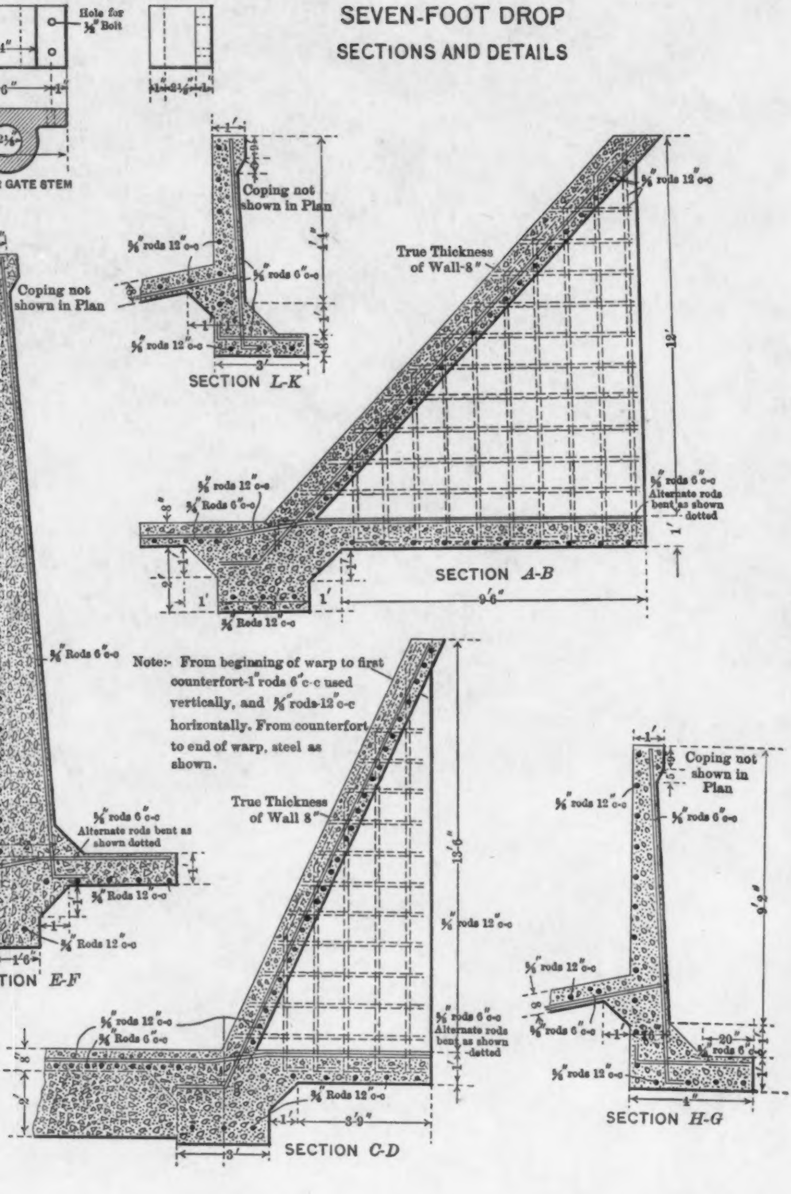
LONGITUDINAL SECTION *M-N*



SECTION *E-F*



SEVEN-FOOT DROP
SECTIONS AND DETAILS



channel with an entirely new twin main canal from a few miles below the concrete head-gate to the mesa ground east of Sharp's Heading. Whatever possibility there was of such fundamental reconstruction and extensions was dissipated by the severe financial stringency in November, 1907.

These designs included the construction of a head-gate on Mexican territory, and the total abandonment of the concrete head-gate, along with diversion, on American soil. The head-gate thus proposed was just behind the levee, about 3 miles below the Boundary Line, with a caisson foundation supporting extensive wing-walls and massive piers, between which there were to be large vertical gates, operated by gasoline motors.

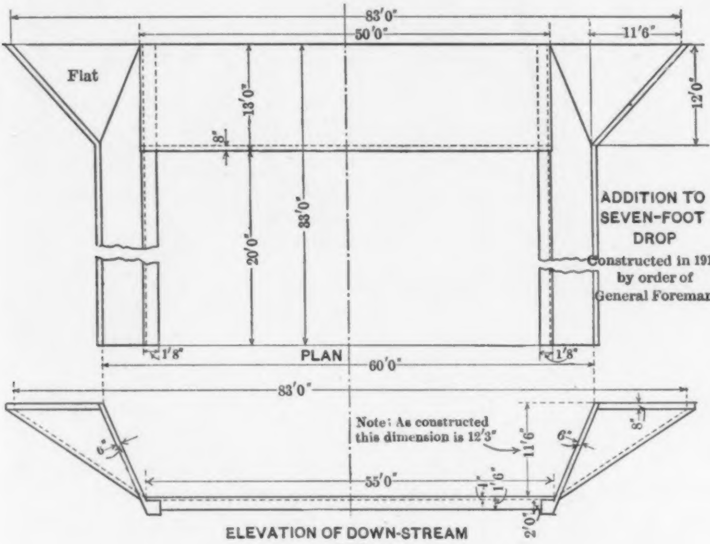


FIG. 32.

Of these designs the only ones constructed were the West Side Main, the Rose Levee and Waste-gate, including the Rositas Heading on the Alamo near Holtville, and the Seven-Foot Drop in the Central Main. The foundations in all cases were merely concrete footings without any sheet-piling, of which the Seven-Foot Drop (Plates CXXVII and CXXVIII, and Fig. 32), and the Rose Waste-gate (Figs. 33 and 34 and Plate CXXIX) are quite typical.

ROSITAS WASTE-GATE

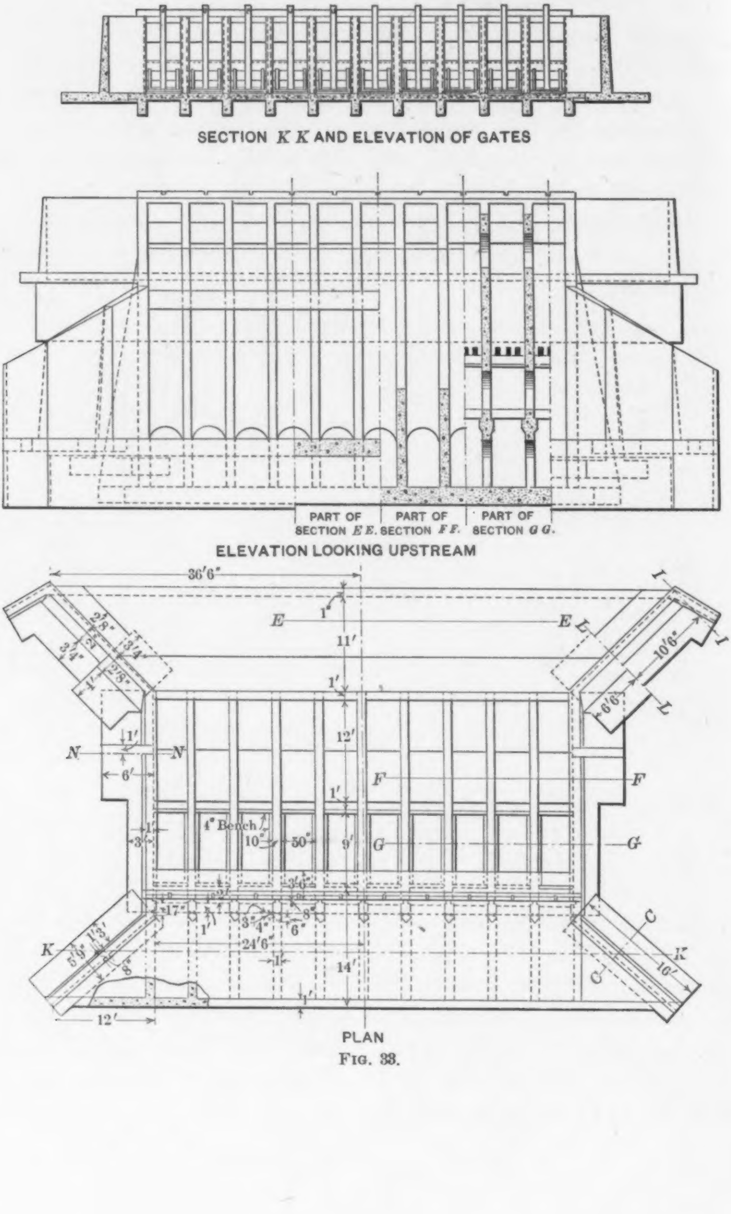


FIG. 33.

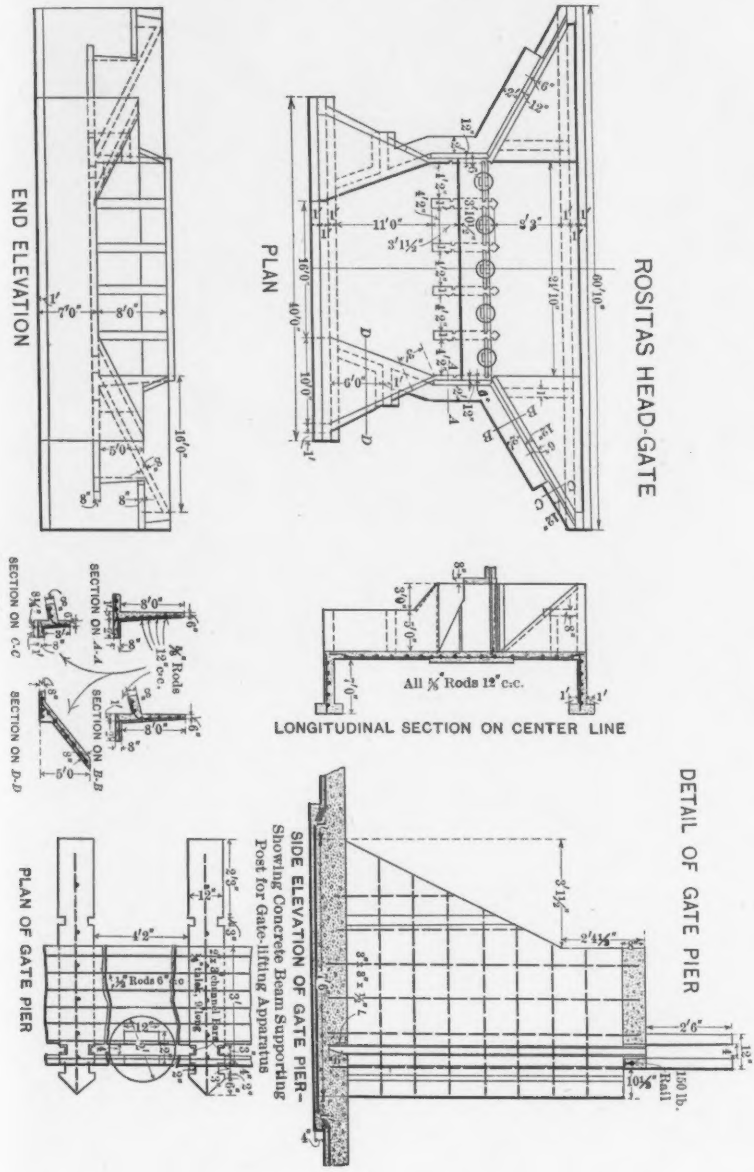

ROSITAS HEAD-GATE
PLAN
END ELEVATION
LONGITUDINAL SECTION ON CENTER LINE
All ⅝" Rods 12" c.c.
SECTION ON A-A
SECTION ON B-B
SECTION ON C-C
SECTION ON D-D
DETAIL OF GATE PIER
SIDE ELEVATION OF GATE PIER—
Showing Concrete Beam Supporting
Post for Gate-lifting Apparatus
150 lb. Rail
PLAN OF GATE PIER

FIG. 34.

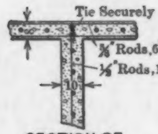
The Rositas Heading and Waste-gate cost \$55 250, and the Seven-Foot Drop, together with the little gates which constitute one structure, cost \$23 750. Concrete work, generally, has cost about three times as much as wooden structures; it has ranged from \$30 to \$35 per cu. yd. for the entire cost of completed structures.

Maintenance and Operation.—Until such time as the promised arrangements for river control and levee maintenance by the Government should be fulfilled, the operations of the irrigation interests, under instructions from higher authorities, were made to include levee maintenance and extension as well as supplying water in wholesale to the mutual water companies. The business done by the corporations was in this way much more varied than that of most irrigation companies, although none of the land they owned was under cultivation.

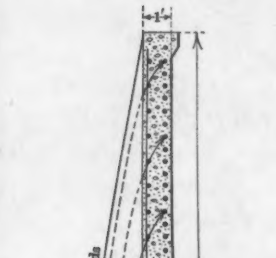
Because of the litigation which seemed inevitable, and indeed which had started ere this, it was deemed essential to have a complete and satisfactory system of accounts. Inquiries were directed to all possible sources of information, including the U. S. Reclamation Service, to discover a system of irrigation accounts similar in a general way to the system of railroad accounts generally adopted and for some time past made obligatory by the Interstate Commerce Commission. No such system was found. The U. S. Reclamation Service has a fairly satisfactory system of construction accounts, but its work, as yet, is almost exclusively construction.

The accounting system, therefore, was worked out, modeled closely after the classified railroad accounts. The account numbers of the Mexican Co. are the same as those of the C. D. Co., except that one thousand is added. Four years' experience with it has resulted in few changes, and in its present form it is extremely satisfactory. Lack of space forbids giving it in full, as there are 146 expenditure and 16 revenue accounts.

In order to ascertain the cost of particular portions of the work—whether new construction, betterments, or ordinary maintenance—special accounts are kept as desired, such as General Manager's Orders (G. M. O's). A G. M. O. is asked to secure authority for, or to secure cost figures on, any particular piece of work, and they are numbered consecutively, beginning with 1 for the C. D. Co. and 1 001 for the Mexican Co. The classified accounts and the G. M. O's are entirely independent, the latter being really a second and additional accounting

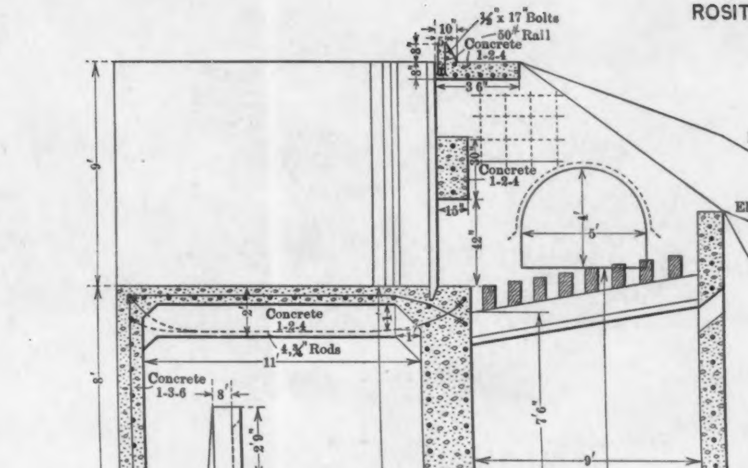


SECTION OF
WALK AND PIER



SECTION *N N*

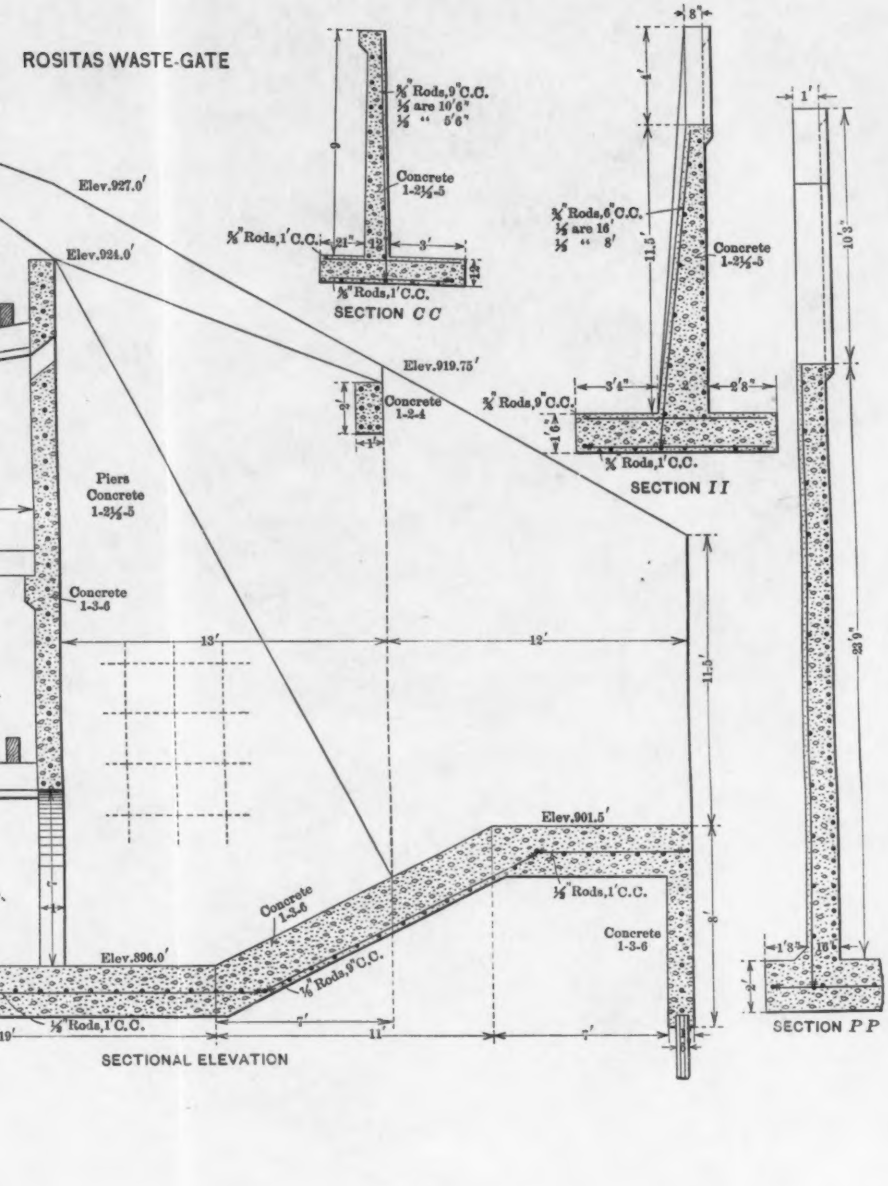
Horizontal reinforcement to be run from vertical rods in front of the buttress, to vertical rod in down-stream end of the adjacent wing wall. All horizontal rods to be hook latched at their ends to make continuous lines through side and wing walls.



SECTION *L L*

PLATE CXXIX.
PAPERS, AM. SOC. C. E.
NOVEMBER, 1912.
CORY ON
IRRIGATION AND RIVER CONTROL,
COLORADO RIVER DELTA.

ROSITAS WASTE-GATE




SECTION OF
WALL AND PIER

for the same expenditure. G. M. O's were never asked for expenditures of less than $300, and were required for all expenditures of certain kinds specified in bulletins issued from time to time.

Distribution was made both to account numbers and to G. M. O. numbers on all payrolls, material requisition blanks, vouchers paying contractors, etc. The auditor's office gathers these figures and makes a monthly report, to the general manager and to the chief engineer, of the expenditures under each account number, totaling under headings. A similar report, of the receipts, under account letters, with totals, is furnished to the general manager. A monthly statement is made up and sent to the general manager and the chief engineer concerning each G. M. O., giving a statement of original estimate, amount expended to date, and percentage completed. Whenever work covered by G. M. O.'s is materially changed, a new G. M. O. is gotten out accordingly.

With such accounts, occasional trips of inspection, to observe the physical condition of the system, will afford a very complete knowledge of the exact condition of the project at all times.

The relationship between the C. D. Co. and the various mutual water companies was at all times very satisfactory and cordial, until the former was thrown into the hands of a receiver, there being one exception, however, in Imperial Water Company No. 5. Differences antedating the beginning of the railroad management and in a large measure due to dissensions between former President A. H. Heber and his Board of Directors, resulted in commencing litigation to compel Imperial Water Company No. 5 to pay up back water rentals. The attorneys of the latter company, in their cross complaint, attacked the validity of the mutual water company and water stock plan of a water right. The suit was before the United States Circuit Court, and in rendering the decision the judge expressed the opinion that the plan was illegal, and, practically speaking, the C. D. Co. was a public service corporation. This was but an opinion, because the State Courts of California have to decide this question finally, as it is a matter of the Constitution and Statutes of the State of California exclusively. The effect of this opinion, however, was to make the Southern Pacific Company feel that it would be unsafe to advance the large sums of money needed to reconstruct and extend the system on the expectation of being repaid ultimately through the sale of additional water stock to cover the new territory which would be brought under the canal. This suit,

as much as any other factor, is responsible for the fact that practically nothing has been done on the betterment and extension work outlined. This litigation with No. 5, although begun in August, 1906, has not yet reached a definite conclusion.

When Imperial Water Company No. 1 was organized, the capital stock was placed at 100 000 shares, and the territory embraced within its limits was 35 000 acres. Of the excess land, a large part was found to be well worth reclamation, so that early in 1910 no No. 1 water stock was available. Imperial Water Company No. 1 was unwilling to increase the capital stock without obtaining from the C. D. Co. a contract increasing the quantity of water it would be entitled to demand and so retain the basis of a 4 acre-ft. per annum per share of stock. It was impossible for the mutual water companies and the C. D. Co. to agree as to the proper division of the "water right" receipts for such 35 000 acres, so no relief was afforded to *bona fide* settlers who had their land ready and had to have water or lose their filings under the rulings of the U. S. Land Department. A trial suit was instituted, known as Thayer *vs.* C. D. Co., which was decided in favor of the plaintiff on March 17th, 1911, by Judge George H. Hutton, of Los Angeles, sitting for Judge Cole in the Superior Court of Imperial County. Judge Hutton in brief decided that the C. D. Co. was a public service corporation; that the rate for supplying water was 50 cents per acre-ft.; that the C. D. Co. was in a position to supply the plaintiff without detriment to the other water users; and ordered that it do so. This decision has been appealed to the State Supreme Court. If it is upheld, the mutual water company will not be a necessary factor in obtaining water from the C. D. Co. This decision has not affected the price of water stock in the valley very materially, because the various mutual water companies own the distribution systems, and the difficulties of getting water from the short mileage of the C. D. Co. main canals through other sources than the distribution systems of the mutual water companies are practically prohibitive. The decision, of course, absolutely precludes extension of the irrigated area any farther through such a water stock plan.

It is very unfortunate that a clause in all the triparty contracts requires the actual seepage and evaporation from each mutual water company's distribution system to be made after a period of 3 years, and that the percentage allowance thus determined should thereafter

be made, to the end that the mutual water companies would pay for the quantity of water which could actually be delivered to the individual settlers. All the mutual water companies joined and selected an engineer, Mr. F. S. Scobey, to represent them in making seepage and evaporation determinations, and did quite a little experimenting and investigating. The engineers of the C. D. Co. also made numerous determinations under the direction of Mr. Herrmann, but for various reasons no experimentation was done jointly. The amount of work and expense involved greatly surprised the representatives of the mutual water companies, and proposals of a compromise were made, one thing helping to this being that such representatives agreed that it would be distinctly unsatisfactory to have different percentages agreed on for each of the mutual companies, as must obviously be the result. The C. D. Co. was thrown into the hands of a receiver just as these negotiations were reaching conclusion on the basis of 6% flat allowance. In the confusion following, the companies insisted on a 10% allowance, and this is the present status.

The results obtained by Mr. Scobey have never been given out, but the experiments by the engineers of the C. D. Co. showed a surprisingly small loss. It appears that the very fine silt deposited in the distribution systems of the various mutual companies forms a practically impervious coating on the sides and bottom, the measured loss in many cases being as small as 0.25% per mile and increasing to a maximum of 1% per mile. From the available experimental data, computation of the losses from the distribution system of Imperial Water Company No. 1, comprising nearly 375 miles of canals, gives the total for seepage and evaporation throughout the year, under present operating conditions, as somewhat less than 7 per cent. The writer has no knowledge of so small a loss by seepage being reported by any authority heretofore.

The experience has been that the mutual water company plan of organization to distribute water, obtaining it from a larger company at wholesale, is highly satisfactory, and it is commended for careful consideration by those who are contemplating irrigation work, if the local laws will permit.

As far as the physical maintenance of the canal system is concerned, it may be said that the average life of the redwood structures, consisting of checks, drops, turn-outs, waste-gates, etc., is about 5

years for the smaller structures and 8 or 10 years for the larger ones. It must be remembered that water is used every day in the year, so that this experience has narrow application. The chief deterioration is caused by a sort of dry rot beginning about 1 ft. below the surface of the ground and extending down not more than 2½ ft. It appears that deeper than that, regardless of the quantity of water present, the heat does not become great enough to cause trouble, while the upper layer of earth is nearly always so dry that the wood is not attacked. In the thin intermediate layer, which is both damp and hot, and perhaps where enough oxygen is available, dry rot appears very rapidly, and wood wet on one side and in contact with earth on the other has the earth side damaged to a depth of from ¼ to ¾ in., sometimes in 9 or 12 months. Redwood subjected to several kinds of treatment has been tried, but with little success. Oregon pine, for that portion of structures covered by earth in that region, rots very rapidly indeed.

The chief lesson taught by the maintenance of the canals—other than that the cost is unusually large because of tule growths and silt—is that inside berms must be avoided and when possible all canals except the sub-laterals should have a double or twin cross-section. In the Imperial Valley the absolute minimum quantity of water is approximately 25% of the absolute maximum—a very unusual condition of affairs. If the sections are identical, it is possible to use one canal for a long time, and have it never less than one-half full, and consequently the velocity of the water is never low enough to deposit the finer silt. This allows a sufficient time for cleaning one canal and then the other every year. When the demand increases beyond the capacity of one canal, both channels are more than half full. In this region, therefore, it is possible with the twin section to control the velocity between the excessive silt-depositing lower limit and the bank-erosion upper limit with absolute certainty. The additional construction expense of the canals for the twin section is much more than justified by the very greatly reduced maintenance charges thereafter.

Duty of Water.—As a basis for estimating the quantity of water required under the Yuma Project, investigations were made, under the direction of the University of Arizona Experiment Station, to determine the water required for various crops in the Yuma Valley. As a result of this work, it was decided that for the average 40-acre unit, 5.8 acre-ft. per annum, measured at the delivery box at one corner of

the field, would be required. Such a figure is exceedingly interesting, but was not obtained under the usual operating conditions. Indeed, the satisfactory delivery of water has been made so recently and in such relatively small quantities in the Yuma Valley as not to justify any definite conclusions.

Water has been actually used in irrigation in Imperial Valley for more than 10 years, and although there seems to be no reason why the duty of water there should not be essentially the same as in the Yuma Valley, the quantity used in the former is only about half as much as indicated by the experimental work mentioned.

The crop census taken by the Zanjeros of Imperial Water Company No. 1 for that district during 1911 is as follows:

Alfalfa	44 262	acres
Barley	28 897	"
Corn	12 034	"
Cotton	6 263	"
Melons	2 153	"
Vineyards	1 352	"
Truck	1 092	"
Asparagus	192	"
Miscellaneous	3 327	"
Total	99 572	"

To supply this acreage, the company bought from the C. D. Co. during the period 274 665 acre-ft. of water, or an average of almost exactly $2\frac{3}{4}$ acre-ft. per acre under cultivation. This is net, after deducting the 10% seepage and evaporation allowance given by the C. D. Co., as already explained. Of this net quantity, according to the water company's report, 92.3% was delivered and charged to the stockholders, making the average quantity of water used, measured at the farmers' boxes, 2.538 acre-ft.

The quantity of water used in irrigation depends on so very many different factors—quality of the land, nature of the crop, proper preparation and leveling of the land, and time of irrigation—that it is only by such general figures covering large areas that much tangible information for engineers is obtained. It must be remembered, however, that the water supplied is charged for on a quantity basis, which

undoubtedly tends to minimize the quantity used, as well as the fact that the farmers know they can have all the water they want at any time they want it, every day in the year. On the other hand, water users—stockholders—are charged their *pro rata* of maintenance and operation expenses, regardless of whether or not they use any water, and also for a minimum of 1 acre-ft. per share of stock. Additional water is 50 cents per acre-ft.

The use of water is increasing in this district, and is in large measure due to the increasing acreage in alfalfa and cotton. The figures are:

1909	........................214 333	acre-ft., net.
1910	........................236 631	" " "
1911	........................274 665	" " "

The figures for other districts are not available, but are probably similar.

The Salt Works Suit.—The New Liverpool Salt Company, whose property was inundated and destroyed on March 8th, 1905, began suit for damages to the extent of $180 000 for land and salt deposits and $30 000 for plant, changing the figures to $325 000 and $75 000, respectively, when the destruction became complete. In July, 1906, a compromise was suggested on the basis of $50 000 cash, but the management of the C. D. Co. declined to consider it. On January 10th, 1908, the case was decided, awarding the Salt Company $456 746.23 damages and $1 500 costs, and a permanent injunction was issued restraining the irrigation companies from diverting more water from the Colorado than would supply the substantial needs of the people residing in the valley. Later, the United States Supreme Court affirmed the decision.

Actions of the Southern Pacific Company.—When this adverse and excessive judgment had been rendered, it was seen that the United States Courts would hold the C. D. Co. liable for all damages caused by the diversion of the river, regardless of the fact that it had occurred in Mexico. All personal property and unsold water stock, therefore, was turned over to the Southern Pacific Company at fair prices, and future payments for water rentals were assigned to the Southern Pacific Company until the moneys loaned by it should have been repaid. At the same time, suit was brought in the United States Court for damages sustained in America, and suit was filed in the Mexican Courts

for damages sustained in both Mexico and the United States, and all real property in the respective countries was attached. The suits in the United States are still pending. Judgment was rendered in the Mexican Courts for $900 000, gold, against the Mexican Co., and enough property of that company was ordered to be sold to satisfy this judgment. Another Mexican corporation was formed by the Harriman interests, and permission to hold the concession of the original Mexican Co. was obtained from the Mexican authorities. At the sale, held on January 28th, 1911, this new Mexican Company bid in all the real and personal property of the Mexican Co., including the concessions from the Mexican Government, for the sum of $325 000, gold, which was less than 40% of the judgment. Thus nothing now remains of the original Mexican Co. except the organization, with a $575 000, gold, judgment against it, and additional suits aggregating nearly $2 000 000 in the Mexican Courts, and absolutely no property.

The new Mexican Corporation, called the Lower California Land and Water Company, owns practically all the parent irrigation company's holdings in Imperial Valley having any value, but has not yet taken possession. Shortly after the sale by court decree, fraudulent dealing was alleged, and on November 18th, 1911, it was advertised that the Judge of the First Instance at Mexicali, would hear any and all complaints in the matter. No one appeared, and it seems probable that the validity of the sale must therefore be confirmed. In that event, the new company will be free of any contracts with the mutual water companies, the C. D. Co., or any one else, and is probably quite beyond the reach of the American Courts. This, however, means little to the water users living in either the United States or Mexico, as the Mexican Government has issued rules and regulations by which water must be sold under the Mexican concession, and these fix the price at 50 cents per acre-ft., and practically in no wise affect the conditions under which American users now receive water.

The C. D. Co., in that event, would be a mere shell; it owns only 65 miles of main canals which produce no revenue whatever, cost no little to maintain, and are hence liabilities instead of assets; it also owns its office, grounds, and buildings, all of which are under attachment, and its liabilities exceed $2 000 000.

Appointment of Receiver.—On December 16th, 1909, the Title Insurance and Trust Company, trustees for the bond issue of the C. D.

Co., applied to the Superior Court of Imperial County to declare the C. D. Co. insolvent and appoint a Receiver, which application was granted. The Southern Pacific Company has bought approximately $325 000 worth of Receiver's certificates, which, together with the major portion of the water rentals received from the mutual water companies, has kept the property going. Application has been made to sell the property, but this has been delayed as long as possible by the attorneys representing the bondholders, the New Liverpool Salt Company, and the old stockholders of the C. D. Co. In a few months, however, it seems probable that this will be accomplished.

Formation of Imperial Irrigation District.—Because of the various difficulties and the serious litigation, the people of Imperial Valley, on July 14th, 1911, by a vote of 1304 to 360, elected to form the Imperial Irrigation District. According to the present law of California, this district can condemn property, even of a public service corporation, and all taxable property within the district is assessable for its needs. It is authorized to incur a bond issue of 50% of the assessed valuation of the property of the district, and the five directors are elected by all voters in the district just as in the case of county and State officials. The assessed valuation of Imperial County this year is $16 161 923; the value of its products is $10 000 000.

It is intended to acquire all the property of the C. D. Co. and the main canals and works in Mexico, giving bonds of the Imperial Irrigation District in exchange therefor. It has not been decided definitely whether the mutual water companies are to be retained, or whether the district is to own and control the entire water system—probably the former will be done.

The present law of California, under which this district was formed, is extremely interesting to water supply and irrigation engineers. It is a considerably changed form of the Wright Irrigation Act, under which, some 20 years ago, a number of irrigation districts were created in California, all of which resulted disastrously. It is believed that in its present form the law is a practicable one, and experience with it will be awaited with much interest.

The Abejas Diversion.

The excessive overbank flow during the summer flood of 1907 started cutting back fingers, as has already been stated. The flood of 1908

continued the work, and made it evident that the deep finger which first would have its grade receded to and through the bank of the Colorado, and thus again divert the entire river to the west, was one of the feeders of the Abejas, and that such diversion would occur about 20 miles below the International Boundary Line.

The situation was carefully watched, and the various interests affected were fully advised of developments. The United States Government had taken no tangible step to repay the moneys expended in closing the second break and in subsequent levee protection work, nor anything definite whatsoever with the Mexican Government looking toward a joint and satisfactory control of the situation. All interests, nevertheless, seemed to feel that the Southern Pacific Company would advance funds to protect the valley when a critical stage should be reached. The writer, in local charge of the situation for that company, had become fully convinced that the truest and best interests of all concerned would no longer be served by the railroad company standing in the breach, and recommended doing absolutely nothing further in protecting the valley than to maintain the existing levee system. In almost everything there comes a time to decline longer to carry the entire load. This recommendation was approved by the higher officials. When the summer flood of 1909 had passed, the expected diversion was an accomplished fact, and as a result the entire low-water flow followed through the Abejas, spread out in a wide sheet without any defined channels, gathered into Volcano Lake and Hardy's Colorado and thence reached the Gulf.

The water in the river at the break dropped somewhat, and, as the river fell to its low-water stage, the water surface for a given discharge was found to be unusually low. The reasons for this have already been explained, but were not then fully understood. The demand for water in the valley increases greatly late in January, on account of the barley crop, and a serious water shortage seemed very probable.

Submerged Weir.—On urgent representations to the War Department, backed by the recommendation of the Reclamation Service engineers, permission was granted in March, 1910, to build a temporary obstruction in the river opposite the concrete head-gate, in order to raise the water a few feet and increase the flow in the canal. Work was started, but the river began rising and rendered it temporarily

unnecessary. When the summer flood receded, in July, the situation was again critical, due to the large requirements in the valley, and work was resumed.

A trestle consisting of 4-pile piers, 15 ft. from center to center, was driven across the river at an angle of about 70 degrees. On this a railroad track was laid, and a little brush and considerable rock was dumped therefrom. Of course, there was no difficulty in developing a head of 2½ ft. This weir or obstruction prevented any danger of water shortage in the valley, but, not being square across the river, it produced eddy currents, just below it on the Arizona side, which cut away the bank to some extent and necessitated considerable expense in bank protection work.

The permission of the Government was given for this construction, on the understanding that it was to be temporary, and would be removed before the next spring flood. In March, 1911, all the piling was blown off, not pulled.

Late in 1910, arrangements were made to obtain the large suction dredge, *Imperial,* and with it in service no further immediate difficulty in diverting sufficient water is anticipated.

A very important fact, however, was developed in the construction and attempted removal of this weir, namely, that the small quantity of rock dumped from the trestles which was required to raise a head of 2½ ft. at low-water stages—about 10 000 sec-ft.—was not undermined and did not settle except to a slight extent in a few places, with the summer floods of 1911 and 1912. These floods were ordinarily large, and, passing over it, had little effect in taking it away. This result was surprising, even to the proponents of rock fill methods of building weirs. The length of time it finally requires to eliminate all effects of this weir from the river flow at that point should be kept track of and reported to the Profession from time to time.

All Parties Frightened.—All the interests in jeopardy were now thoroughly frightened. It was finally realized that the Southern Pacific Company would no longer supply money for river control, and that the diversion was not only an accepted fact, but that the prophesied lowering of the river at the concrete head-gate had taken place. The fear had been that the bed of the river would be lowered at the Abejas break approximately from 5 to 8 ft. and that this lowering would rapidly run back up the river and have such an effect opposite

the concrete head-gate as to prevent diverting enough water to supply the needs of the valley. This fear is now known to have been in large measure unfounded, as the permanent lowering of the water surface at the Abejas probably has not been more than 2 ft., and opposite the concrete head-gate not more than 1 ft., if, indeed, that much at either place.

Another fear was that the Colorado would now discharge directly into Volcano Lake, and during severe flood periods raise this body of water so high that it would flow northward and into the New River channel, thus cutting back a connection, permitting the river again to reach the Salton Sea, but by a course approximately 40 miles longer.

As the summer flood came on, the overflow covering the low lands on each side of the Abejas, especially in the vicinity of Campo Lino, was higher than any existing marks on trees, etc. To prevent this water from getting over the low divide to the north and thence to the Salton Basin, disconnected portions of the remaining gap in the levee line were partly built, the C. D. Co. through its Receiver paying the bills. While the work was in progress, the water, for long stretches, came within a few inches of the top of the fill being thrown up, and was held south of the divide only by strenuous efforts. Probably no very serious results would have followed in any event, although the New River flume of the West Side Main might have been damaged.

The people of Imperial Valley were now thoroughly frightened, and urgent applications were rushed to President Taft, in which the civic and commercial bodies of California, especially Southern California, and the State officials joined. These applications pointed out the inability of American interests in jeopardy to handle a menace originating in Mexico, and the injustice they suffered.

In response to these applications, President Taft sent a special message to Congress, and on the eve of adjournment the two branches of Congress joined in a resolution, approved June 25th, 1910, providing:

"That the sum of $1 000 000, or so much thereof as shall be necessary, is hereby appropriated out of any money in the Treasury not otherwise appropriated, to be expended by the President, for the purpose of protecting the lands and property in the Imperial Valley and elsewhere along the Colorado River, within the limits of the United States, and the President is authorized to expend any portion of such

money within the limit of the Republic of Mexico as he may deem proper, in accordance with such agreements, for the purpose, as he may make with the Republic of Mexico."

On June 8th, 1910, the acting Secretary of the Interior, Mr. Frank Pierce, addressed to the President a communication based on information and recommendations furnished by Mr. Hill, Supervising Engineer of the United States Reclamation Service, advising as follows:

"The ascertainment of what is necessary to be done for the purpose of accomplishing permanent avoidance of these recurring menaces to life and property on both sides of the International Boundary Line will require a thorough examination of physical conditions which, to be effective, should have the co-operation of both governments, and will consume considerable time. In the meantime, unless prompt relief is afforded, a water shortage, if not famine, is probable in the Imperial Valley within the next two months.

"In a country where the heat reaches an intensity of 120° and even higher, the great loss of property and the menace to both animal and human life which may result, should such a catastrophe occur, renders it imperative that prompt measures be taken toward averting the same. To that end, I respectfully recommend that you designate an engineer having familiarity with problems involving river control to proceed immediately with an examination for the purpose of determining whether such emergency exists, and if so, to take the steps necessary to avoid the same."

On the recommendation of Gen. W. L. Marshall, Consulting Engineer to the Secretary of the Interior, John A. Ockerson, President, Am. Soc. C. E., and for many years Member of the Mississippi River Commission, was, on July 19th, 1910, appointed by President Taft. He at once went to Yuma, arriving there on July 30th, by which time Mr. F. L. Sellew, Engineer of the Yuma Project, on Mr. Ockerson's request, had made a survey covering the immediate vicinity of the C. D. Co.'s intake, and had prepared a plat which showed clearly that the bed of the river was above the bottom of the head-gates and that the deficiency of water in the canal was due mainly to the silting up of the intake above and the canal below the concrete head-gate.

There were two possibilities: one was to get dredging machinery into place as rapidly as possible and dredge out the canal; the other was to raise the water in the river by a submerged dam, the latter being temporarily necessary because of the time required to build dredges to do the necessary work. As explained, the weir was begun

in the latter part of July and completed within a month, while the contract was not let for an efficient suction dredge until late in December, the delays in starting the latter being in part due to difficulty in arranging funds therefor, and in part due to a belief in some quarters that, with the submerged weir in place, a dredge was unnecessary.

Mr. Ockerson made an inspection of the Imperial Valley during the latter part of August, and about the middle of that month put out a surveying party which ran a stadia line along the Colorado River from about 6 miles below the Boundary Line to the Abejas break; thence down the dry bed of the Colorado for about 25 miles; down the Abejas River from the point of diversion about 5 miles; and made several cross-sections of the Abejas for the purpose of selecting the best site for a rock fill barrier dam—all of which required about 6 weeks.

On October 4th, Mr. Ockerson reported to the Secretary of the Interior that the Imperial Valley would never be safe from the menace of western diversions due to flood-waters until ample works were constructed to confine such flood-waters to narrow limits along the river proper; that there had been no appreciable lowering of the river bed on account of the Abejas break; that in extreme cases diversions to the west might depress the low-water plane opposite the concrete headgate and render it difficult to supply water to the Imperial Valley with the present diverting works; that the maintenance of levees consists not only in keeping up the cross-section, freeing it from weeds, brush, and burrowing animals, but also in holding in check the tendency of the river to erode the banks and threaten to breach the levees; that a levee located 3 000 ft. from the westerly bends of the stream would probably remain intact for a long time; that if the C. D. Co. levee line had been carried down along the river instead of where it was built in 1907, it would have reached a point 6 miles below the Abejas break, and no break would have occurred there; that completing the upper levee would undoubtedly protect the Imperial Valley from floods for a short time, but constituted only a partial solution of the problem, and even that only temporarily; that, finally, the proper protection of lands in the Imperial Valley required that the Colorado River be restored to its former channel and an effective line of levees be built from a point on the existing levee system about 6 miles below

the International Boundary Line and following along the west side of the stream to a distance of about 3 000 ft. from the westerly bends of the river, approximately 25 miles, where the flood height would be at an elevation below the ground line in the vicinity of Volcano Lake and any diversion of the water would not cause a tendency to flow north.

It was estimated that such a levee would require about 1 300 000 cu. yd. of earthwork and 450 acres of clearing and grubbing; and it was recommended that the top be 8 ft. wide, the slopes 3 to 1, and the berm between the toe of the slope and the edge be 40 ft. wide, and that borrow-pits be on the river side, with traverses 50 ft. wide at intervals of 400 ft.

In commenting on the situation, Gen. Marshall recommended completing the northern line of levees, as originally designed by the writer, to prevent the water from getting north, and providing a suitable and practical intake for the canals of the C. D. Co. He also suggested that, if the present intake (concrete heading) be closed and the Imperial Canal be extended to the Laguna Weir, the matter would be solved, as far as American interests were involved.

A month later Mr. Ockerson went to Washington for a conference, the result being that the work suggested by him was ordered begun. On November 25th bids for the levee construction were opened in Yuma. According to the specifications, the contractor was to assume all risks of interruption of the work by floods, and as the season was by this time far advanced, the prices were deemed too high and endeavors were made to get the work done on force account.

At about the same time it was discovered that the Mexican Government—as would naturally be expected—could not consistently permit the United States Government, or any of its officials acting as such, to perform work on Mexican soil. For six years, the need for making satisfactory arrangements and agreements with the Mexican Government regarding the Colorado River and its control, and proper and equitable division of its waters, had been fully understood, but practically no progress had been made. That fact, however, makes it even more surprising that requests on the part of the United States should have been made to enter Mexican territory and do work therein. The difficulty was very easily overcome, of course, by operating through the medium of a Mexican corporation, as the Southern Pacific had

done, and the C. M. Co., which, as has been explained, is a very large land company, was chosen. Consequently, nothing was done in the name of the United States, but the engineer in charge acted under power of attorney from the C. M. Co., there being a gentleman's agreement between the United States and the C. M. Co.

In this way, on December 12th, contracts were awarded for levee construction, the first 9 miles, aggregating 425 000 cu. yd., at from 19 to 22½ cents; the next 6 miles, aggregating 336 000 cu. yd., at 23 cents; and the remainder, aggregating 325 000 cu. yd., at 36 cents. These figures were afterward considerably increased, the total quantity being 1 277 984 cu. yd., and the total cost, including $20 000 paid for duties, $452 434. The work in the immediate vicinity of the Abejas break and the small quantity of grading on 4 or 5 miles of temporary track was done by the C. M. Co. on force account. President Lovett, of the railroad company, offered to supply, essentially at cost, all the organization, men, equipment, and supplies required for closing the break and for doing any other work that might be deemed necessary.

The Mexican Government had given assurances to the American Minister that duties on stock, material, and supplies would be remitted, which was considerably more than had been done when the railroad had the work in charge. The Government officials, however, were not satisfied with this, and thought that all material should be passed free—a matter very much more difficult to arrange under Mexican laws, as it would require Congressional action, and that country was already in the throes of a revolution. However, after a delay of 2 weeks in getting stock across the line, it was decided to arrange for the duties and depend on a refund later. The contractors began work early in January. At about the same time one pile-driver was put to work on a trestle across the Abejas, and by February 2d (river discharge, 11 000 sec-ft.) the temporary track was completed to and over the trestle, and rock dumping began.

The method was that developed in closing the first and second breaks, and used later in closing the gap in the Laguna Weir, one trestle being deemed quite sufficient, as the maximum head was not expected to exceed 7 or 8 ft. At the Andrade quarry there were approximately 15 000 cu. yd. of rock ready to load with steam shovels, and the quarry was well developed, having a face about 900 ft. long and averaging 40 ft. high. Two 4-cu. yd. steam shovels were secured from

the Southern Pacific Company, and a 2½-cu. yd. shovel of the Reclamation Service was brought down from the Laguna Weir. Work trains, men, and "battleships" were obtained from the railroad, as required, and rails and track material were furnished on a rental basis.

On February 7th, a sudden rise (maximum discharge, 23 000 sec-ft.) caused a breach in the trestle; this was closed 10 days later. On March 7th another small rise in the river carried away seven bents, and 6 days later another rise (maximum discharge, 35 000 sec-ft.) brought down a mass of drift and wrecked a considerable length of the bridge, caused the loss of a pile-driver and one steel "battleship," and drowned one man. On the 28th the pile-driving was resumed and the operations were continued without further mishap until May 15th, when work on the dam was shut down.

In making this closing, the rock fill was not kept at uniform height for the entire length of the trestle, the overpour for some of the time being confined to three places with a total length of from 260 to 275 ft. It is probable that this explains in large measure the breaking of the trestles by floods and drift, after obtaining an effective rock mattress, such as is provided by dumping two or three cuts of "battleships" all along. In building the Clarke Dam, floods (maximum discharge, 32 000 sec-ft.), with heavy drift, threw the trestle out of line in only one place after rock dumping began, and caused no other damage.

The number of cars unloaded in the Abejas closing work is given in Table 22.

TABLE 22.

Period.	"Battleships."	Flats.	Dealey.	Dinky.
January 17-31	83	....	....	12
February 1-10	314	....	....	....
" 11-20	350	5	....	....
" 21-28	616	....	....	34
March 1-10	181	....	....	....
" 11-20	108	....	....	....
" 21-31	207	....	....	108
April 1-10	747	43	....	20
" 11-20	781	40	6	....
" 21-30	581	70	37	....
May 1-2	41	3	....	8
Totals	3 996	161	43	182

The total quantities were: 139 860 cu. yd. of "battleship" rock, 193 cu. yd. of flat-car rock, 516 cu. yd. of Declez rock, and 1 092 cu. yd. of quarry rock in dinky cars, a total of 143 400 cu. yd. up to May

2d. The total quantity of rock used to May 15th, when work closed down, was about 180 000 cu. yd. The total cost of the dam is given as $347 500.

About 140 000 cu. yd. of rock were used before the water was practically shut off in making this closing. The reasons for requiring such a large quantity probably are that relatively little flat-car rock of large size was used, the fact that the rock fill was not carried along at a uniform height for the entire length of the overpour, and the slow rate at which the rock was unloaded. The first methods of quarrying were not well adapted for giving the maximum output, consisting of operations along the top of the rock mass by the edge of the quarry face, but, later, horizontal tunnels were driven into the quarry face at intervals along the bottom and large charges of explosives were used, after which the output was much increased.

The levee work went along very rapidly, the contractors fortunately encountering no flood difficulties or delay, and on April 7th the last of the grading outfit left the work.

While operations were in progress, the Revolution in Mexico began, and resulted in the abdication of President Diaz. On February 21st, the Revolutionists captured Algodones. On April 16th a large body of Mexican Federal troops arrived at the break and remained guarding the work from interruption until May 10th.

Damage to the Work.—The 24.6 miles of levees were constructed in accordance with the recommendation, namely, with a top width of 8 ft., side slopes of 3 to 1, borrow-pits on the river side 400 ft. long, with intervening uncleared traverses 50 ft. wide, a berm width of 40 ft., the entire ground covered by the levee and the berm cleared, and roots and stumps grubbed, and a muck-ditch, of such depth as would reach through the adobe soil under the construction, dug out and filled with good material under the axis of the levees. Except for a very few cases of logs and brush in the levee section, and inefficient muck-ditching reported to have been disclosed where the levees were broken, the dikes were very well constructed in accordance with the specifications. The levee was built to a grade of "5 ft. above the high-water marks of the 1909 summer flood," chiefly for the purpose of having excess material wherewith to remedy deterioration, rather than through any fear of overtopping from floods." No railroad track or gravel blanketing was put on the levees, because of the

belief "that it would be better to extend the levees as far as practical, rather than dissipate available funds for mere convenience of maintenance."

A low levee was built along the south side of the break, extending from the dam a short distance up stream, to prevent water from getting into the levee borrow-pits until the grading outfits had finished work in the vicinity. As the water in the river was gradually raised, partly by increased discharge (total, 19 000 sec-ft.) and partly by increasing the height of the rock fill across the break, this low levee was, on April 21st, overtopped and almost at once about 1 000 sec-ft. of water started down the borrow-pit clearing, about 2 000 sec-ft. going into the old Colorado River channel. When the water hit the uncleared traverses it cut the berm and side-swiped the levee at almost every traverse for several miles. Work on the rock fill dam was stopped, and the men were set at work protecting the levee. Later in the day this was stopped, and work on the dam was resumed. The latter was facilitated materially by the waters breaking over into the borrow-pits, the elevations of the water surface above and below the dam being quickly changed from 79.6 and 71.6, respectively, to 78.6 and 71.0, the depth of overpour being reduced 1 ft. and the head on the structure 0.4 ft. By dumping rock and filling the holes where the confined overpour occurred, the situation there was soon in hand, and 8 days later (April 29th) the elevation of the lowest point of the dam was 80.6, or 1 ft. higher than the water surface up stream, where the water broke into the borrow-pits, and the flow over the structure was stopped.

By that date the levee to the south was cut entirely through in several places in the first 6 miles, and it was evident that the water would soon merely detour around the dam and continue to follow the Abejas channel below. The river discharge then was 21 900 sec-ft., and about 4 000 sec-ft. were going down the old channel, the remainder running through the levee breaks and toward the west. On May 7th, the discharge of the river had increased to 32 800 sec-ft., of which perhaps 9 000 sec-ft. were running down the old channel of the Colorado, while the water varied in height from 4.3 to 6 ft. below the top of the levee in the 2 miles immediately north of the dam.

The rock fill was then up to the track all across, and the total percolation through it was reduced to about 120 sec-ft. The eddy

currents below the dam, 9 days later, weakened the earth fill about 300 ft. from the north end of the dam, until the water broke through, and in a few hours the entire discharge of the river (except a little overbank flow) was going through it and down the Abejas. Soon the earth fill on the south was cut through, and thus the rock fill dam was made an island in the Abejas channel, which is the situation at present.

The final injury on the levee work prior to the summer flood was three breaks and several places side-swiped on the north levee, totaling about 16 000 cu. yd., and thirteen breaks, varying from a few hundred feet to more than 2 miles in length, and much side-swiping in the first 8 miles of the south levee, totaling about 200 000 cu. yd., or about 50% of the original earthwork in this stretch. The fact that such injury was caused by so small a quantity of water reaching a maximum depth of only 4 ft. on the levee shows clearly the ease with which the material of the region is eroded.

The protective measures used were sand bags and brush placed to check erosion and the dynamiting of the traverses which, with the drift their vegetation caught, deflected the water to the levee section. The latter procedure had the bad effect of converting the borrow-pits into a continuous canal, but with severe eddy currents caused by the remains of the traverses. These endeavors had little effect; indeed, to hold long stretches of embankment against such action is practically impossible.

On June 1st, Mr. Fisher, Secretary of the Interior, called a Board of Review consisting of Mr. F. H. Newell, Director of the U. S. Reclamation Service; Gen. W. L. Marshall, Consulting Engineer to the Secretary of the Interior; J. B. Lippincott, M. Am. Soc. C. E., formerly a Supervising Engineer of the U. S. Reclamation Service and now an Assistant Engineer of the Los Angeles Aqueduct; Mr. C. E. Grunsky; Mr. J. A. Ockerson, Engineer in charge of the work; and Gen. Harrison Gray Otis, President of the C. M. Co., to report on the work done under the appropriation. All the members of the Board, excepting Gen. Marshall, have examined the territory and understand the situation fully, although none of them except Mr. Ockerson has been on the ground since the Abejas diversion occurred.

On June 7th this Board made a report based on information as to the recent work done and results obtained, supplied by Mr. Ockerson,

and answering specific questions submitted by Secretary Fisher. The full text of this report is as follows:

"1.—A breach in the west bank or levee of the Colorado River, if made at or within a mile south of the California boundary, or south of Mile 18, below Yuma Bridge, will result in water flowing directly into the drainage areas of the Alamo and New Rivers and thence into Salton Sea, which would be disastrous to property in the United States.

"2.—A breach in the river bank at any point between Miles 18 and 55, below Yuma Bridge, will result in spreading water over the Delta of the Colorado River, with a flow into New River *via* Volcano Lake, menacing Imperial Valley.

"3.—(*a*) The best practical method for the protection of land and property in the United States against a discharge directly into the Imperial Canal and thence through Imperial Valley into Salton Sea, is to protect and maintain the levees as at present located for a distance of at least 10 miles south of the California boundary and to hold the river by adequate bank revetment practically on its present alignment. (*b*) This levee, if extended to a point opposite the south boundary of Arizona, or about Mile 27, will also prevent a discharge directly into the Paredones.

"4.—(*a*) As a remedial or precautionary work to prevent damage which might result from a crevasse directly into the Imperial Canal or Alamo River, we have considered a secondary levee west of the river levees, across Imperial Canal, and large channels leading to the natural depressions, or diverting works, conducting the water southwesterly into channels leading into Volcano Lake, but it is believed that the cost of any such secondary defence could be better expended in maintaining the main line of defence at the river. (*b*) As a necessary defence against the northerly flow of any water reaching Volcano Lake, whatever be the treatment of the Lower Colorado River, there should be an embankment well protected against wave wash on its south slope, constructed about on the line of the levees already built extending from high ground north of Volcano Lake to a connection with the levees already built by the California Development Company, southwesterly, toward this region from the Colorado River. The top of this embankment should at its western end be not less than 10 ft. higher than the rim land at Volcano Lake. This embankment is an essential requisite as a protection of Imperial Valley, against menace from the south, and should be constructed without delay.

"5.—(*a*) The maintenance of the works constructed in 1906 and 1907, closing the breaks of the Colorado into the Alamo, and the maintenance of these and of the river levees since constructed as far south as the head of the Abejas are essential requirements. Suitable arrangements for their repair and maintenance should be made with

Mexico through the proper authorities. We do not consider the immediate closure of the break into the Abejas and the reconstruction of the levees below the break as essential to the protection of property in the United States. The ultimate treatment of this section of the Colorado River in co-operation with Mexico may well be determined by negotiations between the governments of the two countries. As a feature of the permanent solution of the river problem it is desirable that the Abejas break be closed, that the levee constructed in 1911 be repaired and maintained, and the Colorado River restored to its former course. (*b*) Provided the water of the Colorado is discharged into the Gulf of California through the Abejas into the Pescadero and Hardy Rivers, there is little probability of the cut back affecting the Laguna Dam. Such cut back will not injuriously affect the heading of the Imperial Canal or levees adjacent thereto, with a possible exception of requiring the lowering of the intake of the Imperial Canal a few feet. This is not a serious matter, and is one that should be dealt with by the California Development Company itself when necessary. The diversion by the California Development Company should be facilitated during low-water stages by dredging, or by lowering the sill of its intake, rather than by placing obstructions in the channel of the river below the intake.

"In view of the existing emergencies along the Colorado River, arrangements should be made with the government of Mexico to provide for the early creation of an International Colorado River Commission, embracing in its membership both American and Mexican engineers, invested with large powers and ample authority to examine into and to submit a basis for the adjustment of all questions relating to the conservation, use, and control of the waters of the Colorado River with a view to such governmental action as shall result in a complete, just, and final settlement of all such matters at issue between the two nations. We recommend that further work should be undertaken at once and in approximately the following order:

"(*a*) The levees north of Volcano Lake should be raised, strengthened, and extended.

"(*b*) The existing levees along the west bank of the Colorado River to the Abejas should be repaired and protected. For this purpose and to meet emergencies, there should be immediately available the sum of at least \$1 000 000. This sum provides only for the temporary maintenance of levees, and does not include the systematic revetment of the river banks.

"The conference ventures to suggest certain international questions which are involved and which will inevitably have to be met sooner or later:

"(*a*) The matter of the permanent protection of existing works on both sides of the international boundary line, the construction of fur-

ther works, and the conditions under which the present and future projects may be carried out on Mexican soil, with the consent and co-operation of the government of Mexico for the benefit of both countries, to the end that the greatest practicable quantity of water of the Colorado River may be made available for irrigation by means of storage reservoirs and otherwise, and the least possible quantity be permitted to flow unused to the sea, and to what extent the cost of such maintenance should be chargeable to properties benefited and to what extent chargeable to either government.

"(*b*) That permanent agreements with the government of Mexico shall be entered into, having in view the just apportionment of the waters between the two countries, irrigation to be paramount to navigation.

"(*c*) The method by which either nation may acquire rights of way for canals, levees, and related works, each within the territory of the other, and the authority to maintain such works.

"(*d*) The modification of the boundary line between the United States and Mexico with a view to facilitating the solution of the entire Colorado River problem. An authoritative, just and final determination of this important question, now a matter of public discussion, will have the effect of removing existing doubts in the public mind and of settling the matter for the benefit of all concerned.

"The members of the conference desire to call attention to the fact that the plan and execution of the work accomplished during 1911 followed well established principles of good engineering. That so large an amount was accomplished in such a brief space of time, under adverse circumstances, is worthy of the highest commendation. That the restoration of the Colorado River to its former channel was not realized is chargeable to the delay in the negotiations, which prevented prompt inauguration of the work and the prosecution during the low-water season, and also to the disturbed political situation and strike which demoralized labor conditions. The members of the conference, in addition to the conclusions above reached, present also a statement of physical and related facts embodied in an abstract of the data available, and found largely in the reports of J. A. Ockerson and of C. E. Grunsky. Also in the printed hearings before a subcommittee of the Senate Committee on Claims, referring to Senate 4170, January 21, 1909."

When President Taft received the report, he sent a special message to Congress recommending the appropriation of another $1 000 000 to continue the control work, but Congress failed to comply. It is desired to call particular attention to the last part of this report containing Suggestions *a, b, c,* and *d* regarding certain international questions involved, which sooner or later must be met. These suggestions

contain the crux of the entire situation, and it is to be hoped that they will be expeditiously followed out. It is perhaps desirable to call attention to the wording of Suggestion *d* regarding modification of the Boundary Line between the United States and Mexico, the suggestion being implied that the matter be taken up more with a view of putting a quietus to the proposition, than with the idea of obtaining any territory from the Mexican Government.

Surveys by the New Mexican Company.

As soon as the summer flood of 1911 began to recede, and the extent of the damage sustained by the new work was ascertained, the Harriman interests controlling the new Mexican Company deemed it wise to ascertain how effective the line of levees north of Volcano Lake would be in holding back the waters of a very large summer flood. Accordingly, three surveying parties were put out early in July under the immediate direction of Mr. Hind. The surveys were carried down the river to The Colony and as far west as Volcano Lake, much of the territory being covered quite thoroughly. The field work was completed during the latter part of September, and the data were assembled and mapped. Mr. Randolph analyzed these data and compiled a report which was forwarded to Mr. R. S. Lovett, Chairman of the Executive Board in New York City, recommending:

1st.—That the westerly portion of the Volcano Lake levee be raised and extended so as to occupy a plane 6 ft. above the then present crest; and

2d.—That an effective levee system, including a rock dam shutting off the Abejas diversion, be constructed along the river to a distance of 16 miles below such dam; that the entire levee should be blanketed with gravel; and that a railway track be laid thereon in good condition for further operation when necessary.

Mr. Lovett, on December 13th, 1911, made a formal offer to President Taft, on behalf of the Southern Pacific Company, to return the Colorado River to its original channel and to maintain the levees necessary to keep it there for one year, providing the Southern Pacific Company be repaid for the work done in 1906-07 at President Roosevelt's request, in the sum of $1 830 673.90, being the amount reported as proper under the circumstances by Mr. Grunsky in his statement to the Committee on Claims during 1908; and provided,

further, that an additional appropriation of $1 500 000 be placed at the disposal of the President of the United States with which to pay the actual cost of the work to be done and the cost of maintaining it for one year, the Southern Pacific Company to stand any excess of cost over and above such an amount; that the transportation charges of the Southern Pacific Company in connection with the work should be in accordance with the arrangements in effect during the work under Mr. Ockerson's direction; and providing, further, that should the Southern Pacific Company fail to return the Colorado River to its former channel and to retain the levee for one year thereafter, then in that event the Southern Pacific Company should receive no compensation or reimbursement for the work which it would do under that offer.

This proposition was referred to Gen. Marshall, Consulting Engineer for the Department of the Interior, who, under date of January 5th, 1912, reported in substance that the work proposed should not be done either by it or by any other agency on behalf of the United States at this time, nor until the entire subject of improving the Colorado River and utilizing its water should be investigated by an International Committee representing both the United States and Mexico. The following day, the Secretary of the Interior, W. L. Fisher, forwarded Gen. Marshall's report to President Taft and approved of its conclusions, stating that the suggestions constituted the most important recommendation of the Advisory Board of June 7th, 1911, and adding:

"I consider it of great importance that negotiations should be immediately opened and vigorously conducted with a view of arriving at a treaty with Mexico covering this subject."

President Taft, in his message to Congress a few days later, placed the whole matter before Congress without recommendation.*

Perhaps the most important information obtained by these surveys is the fact that the average elevation of the bottom of Volcano Lake is 28 ft. while the general average prior to 1907 was 17.8 ft. above sea level. In other words, the bottom has been raised 10 ft. since the Colorado began to flow directly into the lake, during which time it has discharged into it approximately 30 000 000 acre-ft. of water. The streams below, draining to the Gulf, are now normally clear, showing that most

* See also House Document 204, 62d Congress, Second Session.

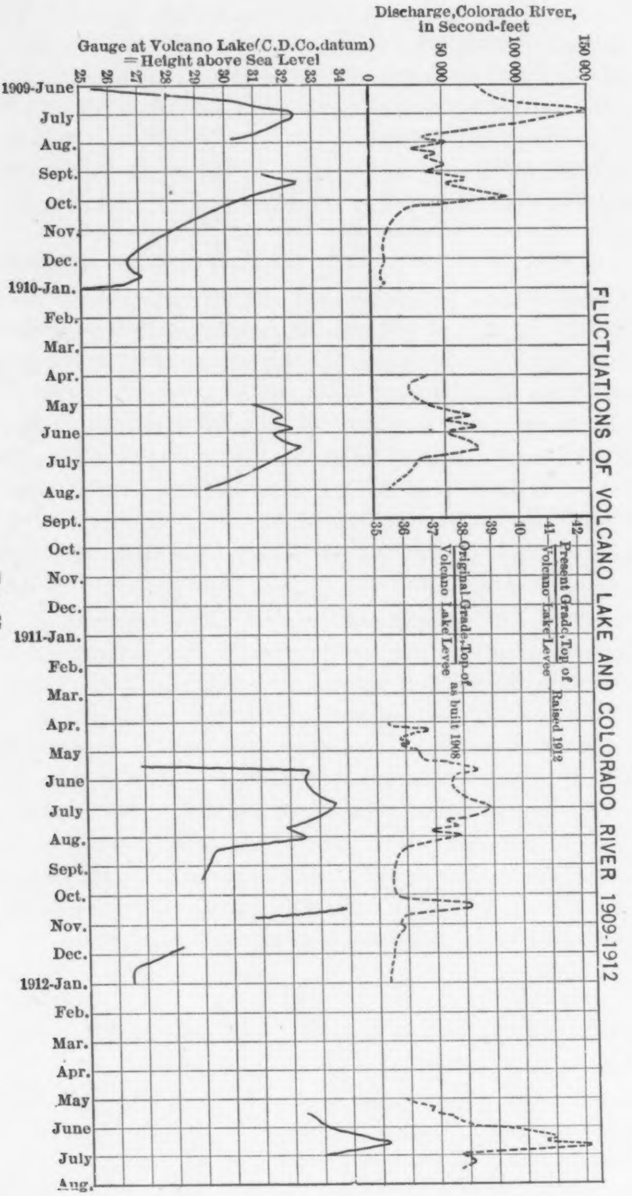

FLUCTUATIONS OF VOLCANO LAKE AND COLORADO RIVER 1909-1912
Gauge at Volcano Lake(C.D.Co.datum)
=Height above Sea Level
Discharge,Colorado River,
in Second-feet
0
50 000
100 000
150 000
25
26
27
28
29
30
31
32
33
34
35
36
37
38
39
40
41
42
Present Grade,Top of
Volcano Lake Levee
Raised 1912
Original Grade,Top of
Volcano Lake Levee
as built 1908
1909-June
July
Aug.
Sept.
Oct.
Nov.
Dec.
1910-Jan.
Feb.
Mar.
Apr.
May
June
July
Aug.
Sept.
Oct.
Nov.
Dec.
1911-Jan.
Feb.
Mar.
Apr.
May
June
July
Aug.
Sept.
Oct.
Nov.
Dec.
1912-Jan.
Feb.
Mar.
Apr.
May
June
July
Aug.

FIG. 35.

of the silt content of the water reaching the lake is being dropped over its bottom, together with some material eroded from the new channels formed by the diverted river. On the other hand, very much suspended material carried by the waters is let down before reaching the lake. The conditions are extreme, yet they indicate the extreme silt deterioration which may be expected in reservoirs on the Gila, Salt, Verde, Colorado River below The Needles, and similar streams.

Improvement of Volcano Lake Levee and Repair of River Levee.

Contracts were let on January 4th, 1912, for raising the Volcano Lake levee 3½ ft. and widening the crown to 12 ft., 155 000 cu. yd. at 25 cents; repairing the recently constructed levee along the river north of the Abejas break, the contract calling for 15 000 cu. yd. at 22 cents; and paving with rock the south or water face of the Volcano Lake levee, approximately 70 000 cu. yd., for $1.50 per cu. yd. This rock was obtained from the mountain sides at or near Cerro Prieto. The temporary railroad track from the C. D. Co. levee to the Abejas break was taken up, and the track material returned to the Southern Pacific Company.

Assistant Secretary of the Interior Thompson went to the City of Mexico to make arrangements whereby the Government might do this work in Mexico, but was unsuccessful. Until the United States concludes arrangements for working in conjunction with the Mexican Government, operations on Mexican soil will doubtless have to be handled in a roundabout way, particularly as long as there is fear of complaint that lawlessness interferes and delays the progress of such work as may be permitted. Mexico, however, authorized the foregoing work to be done by the C. M. Co. through the engineer assigned by the United States, Mr. Ockerson, and a gentleman's agreement was reached between that company and the United States.

Conclusion.

Because of the various successful and unsuccessful work done in the region, the engineering features of irrigation and river control along the Lower Colorado are now understood, and engineering construction methods are thoroughly developed. The successful attempts in closing breaks along the river with rock fill barrier dams according to the method developed during the first and second closings have

standardized this class of work. The Southern Pacific Company can easily, on very short notice, furnish all the equipment, men, and organization needed to do all the various classes of work involved, directly or indirectly, in controlling the river. The essential features of successful levee construction there have been made very clear. The maintenance and operation of the irrigation canals involve caring for excessive quantities of silt, and sufficient data regarding the silt problem in the Main Canal have not yet been obtained to decide on the most economical method of diversion.

The Colorado River Delta now presents no unusual unsolved engineering difficulties; its problems are chiefly matters of statecraft in both river control and irrigation. At the conclusion—in the near future—of existing litigation in American Imperial Valley, irrigation of the territory will be notably free from legal and managerial entanglements. This will be as soon as, and not until, reasonable treaty provisions between the two nations are arranged. Such a treaty is indispensable for the proper handling of river control work. Fortunately, both Governments profess not only willingness, but impatience, to adjust the matter.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

PREVENTION OF MOSQUITO BREEDING.

By Spencer Miller, M. Am. Soc. C. E.

To be Presented December 18th, 1912.

In order to promote the health and comfort of the public, mosquito breeding must be prevented as far as practicable in many parts of the United States.

Mosquitoes breed in standing or quiescent water. In the summer of 1910, a pest of common house mosquitoes invaded a city. These mosquitoes bred chiefly in the water standing in the sewer catch-basins during the summer drought. During periods of dry weather, these catch-basins are now regularly oiled, and the mosquito pest has been largely reduced. When these basins are flushed by heavy rain-storms, mosquitoes are not produced for about 10 days. Can a practical type of sewer catch-basin be constructed so that it will not breed mosquitoes?

Sewer manholes provided with a pail suspended just beneath the manhole covers have been found to be extensive breeding places for mosquitoes. Can this pail be eliminated, or if not, can it be made to drain off the water?

Engineers who direct the construction of great railroad fills across salt-marsh areas frequently neglect to provide for the adequate drainage of these marshes. The culverts which are provided are almost invariably placed too high to drain the lands sufficiently to pre-

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

vent the breeding of salt-marsh mosquitoes. The latter lay their eggs in mud, and they are hatched as soon as the mud is well flooded. Six broods of these mosquitoes (within the vicinity of New York) occur each season. Each square foot of salt marsh may breed 5 000 mosquitoes in a single brood. An acre of land contains 4 840 sq. yd.; thus, 1 sq. ft. of salt-marsh breeding area will furnish one mosquito for every square yard in an acre of land. Surely 5 000 mosquitoes to an acre must be regarded as a "pest." A square mile contains 640 acres. A pool on the salt marshes, about 25 ft. square, may supply a pest of mosquitoes for 1 sq. mile of territory; 100 000 people could live on 1 sq. mile of land. Such a pool, therefore, could breed mosquitoes enough to torment 100 000 people for six periods during a season; and, furthermore, since mosquitoes may live two weeks, these 100 000 people may be tormented for one-half the summer season. When so great a pest may proceed from so small a source, is it not the duty of engineers employed in railroad work to give full consideration to drainage facilities for the salt marshes which are being crossed?

In other ways, also, the work of engineers may be directly associated with the prevention of mosquito breeding. It was established in the City of New Orleans that the yellow fever mosquito bred most prolifically in elevated water tanks and water barrels. By the introduction of a central water supply system, these water tanks and water barrels were eliminated, and the mosquito pest was thereby largely reduced.

Cesspools are prolific breeders of the common house mosquito. The introduction of a sewer system and the filling up of these cesspools have been known to reduce largely the mosquito pest.

City engineers can order dirt taken from roads in grading to be dumped in low spots where water stagnates.

The civil engineer has an opportunity to serve his fellow men and do a vast amount of good work in the prevention of mosquito breeding, if he acquires the knowledge now available on their breeding habits.

It is not intended in this paper to review the work which has been accomplished by entomologists during the last few years. A number of pamphlets, devoted exclusively to the subject, have been published by the Department of Agriculture, and are obtainable for the asking.

Reports by State Entomologists are also frequently obtainable. Dr. L. O. Howard, Chief Entomologist of the Department of Agriculture, has published an extremely valuable book on the subject, which is obtainable from publishing houses. The late Dr. John B. Smith, State Entomologist of New Jersey, prepared a most elaborate review on the mosquitoes bred in the State of New Jersey, which, if not out of print, is obtainable by application to Dr. Lipman, Director of the State Experiment Station, New Brunswick, N. J. Dr. Ethan P. Felt, State Entomologist of New York, has prepared a review on the mosquitoes of New York State, which is intended for free distribution. There are numerous other books on the same subject, many of which are printed abroad.

The writer quotes herewith in full the "Mosquito Brief" prepared, a few years ago, by the American Mosquito Extermination Society, with the aid and endorsement of the members of the Advisory Board of Entomologists. This Brief may be regarded as the A-B-C of the whole problem of mosquito prevention, and is authoritative.

"Mosquito Brief.

"1.—There are over 100 species of Mosquitoes in the United States.

"2.—Mosquitoes breed only in water. They may breed in any kind of quiet water unstocked with destroying fish.

"3.—Mosquitoes generally require from one to three weeks to develop from eggs to winged insects in warm weather, longer in cold weather. Some female mosquitoes three days old lay eggs, the average is greater. Some species lay as many as three or four hundred eggs at once, some lay them singly. Mosquitoes may live several months (as shown by hibernation and otherwise), but probably few live over a month.

"4.—Mosquitoes do not breed in grass, but rank growths of weeds or grass may conceal small breeding puddles, and form a favorite harboring place for adults. The Pitcher Plant holds sufficient water to breed a rare and small species.

"5.—Different species of mosquitoes have as well defined habits as different kinds of birds, flies, etc. Some are Domestic, some Wild, some Migratory.

"6.—Most Domestic Mosquitoes breed in fresh water, fly short distances and habitually enter houses.

"7.—Most Migratory Mosquitoes breed in salt and brackish marsh areas, fly long distances. They are not conveyers of malaria.

"8.—Rigid tests, both direct and eliminative, have proved that certain species of mosquitoes are the only known natural means of

transmitting malaria and yellow fever. Some other diseases are known to be conveyed by mosquitoes.

"9.—Of the domestic varieties, the dangerous malarial mosquitoes (several species of the genus Anopheles), are among the most generally distributed. They seem never to travel far, only a few hundred yards.

"10.—A most common and dangerous domestic mosquito in the South and the tropics is Stegomyia fasciata, which is the natural conveyer of yellow fever.

"11.—Mosquitoes are known to bite more than once, as can be seen by observation and is proved by the transmission of disease from an infected person to a new subject.

"12.—Mosquitoes are a needless and dangerous pest. Their propagation can be largely prevented by such methods as drainage or filling of wet areas, removal, emptying or screening of water receptacles, spraying standing water with oil where other remedies are impracticable. Attention should be paid to cisterns, house-vases, cesspools, road basins, sewers, watering troughs, roof gutters, old tin cans, holes in trees, marshes, swamps and puddles. As malarial mosquitoes may be bred in clear springs, the edges of such places should be kept clean, and they should be stocked with small fish. The breeding and protection of insectivorous birds such as swallows and martins, should be encouraged. Thorough screening of houses and cisterns is necessary to prevent the spread of malaria or yellow fever. The continued breeding of any kind of mosquitoes with the attendant menace to public health and to the life and comfort of man and beast is therefore the result of ignorance or neglect."

The writer's own observations, which cover the past eleven years, have seemed to establish the fact that the most troublesome varieties of mosquitoes are those bred in man-made water holders. Common house mosquitoes habitually enter houses and are busy all night singing and stinging. When these mosquitoes are ready to lay their eggs, they leave the house for the nearest spot where standing water is to be found. They lay their eggs in rain-water barrels (man-made water holders); in cesspools (made by man); in rain-water gutters which are not kept clean; in fire-pails; in pools formed in excavations, cellars, trenches, etc. These man-made water holders are prolific breeders chiefly because they breed nothing else. Natural pools which hold water for a greater part of the season frequently contain water beetles and other creatures which devour the mosquito larvæ, and thus prevent their development into winged insects.

Another fact established by the writer's observations is that it pays to eliminate breeding places throughout small areas, and that the

greater the area thoroughly controlled the better will be the results. For example, he observed one section where no mosquitoes were seen for two weeks, while people living less than 400 ft. therefrom were suffering severely from the pest. An examination showed that where the mosquitoes were troublesome, two water barrels were found breeding extensively.

Another example which came under his observation, was that of a municipality, 2 miles square, which was known to be thoroughly controlled, but was found to suffer considerably from common house mosquitoes which were breeding extensively beyond its borders.

Because of these experiences, he believes that the unit of operation against mosquito breeding should be enlarged. The county becomes a proper sized unit; furthermore, all counties throughout a State should work in unison in order to accomplish the best results. A law passed by the State Legislature of New Jersey during the session of 1912 is printed herewith. It provides for the appointment of commissioners by the Supreme Court Judge of each county throughout the State. It also provides for raising, by taxation, the funds needed to carry on the work of mosquito prevention.

"CHAPTER 104.

"An Act for the establishment of county mosquito extermination commissions and to define their powers and duties.

"BE IT ENACTED *by the Senate and General Assembly of the State of New Jersey:*

"1. In any county of this State it shall be the duty of the justice of the Supreme Court presiding over the courts of said county to appoint six persons, three of whom must be persons who are or have been members or employes of boards of health. A board of commissioners to be known as 'The County Mosquito Extermination Commission,' inserting the name of the county in and for which the commissioners are appointed. The commissioners first appointed under the provisions of this act in any county shall hold office respectively for the term of one, two, and three years, as indicated and fixed in the order of appointment, and all such commissioners, after the first appointment, shall be so appointed for the full term of three years; vacancies in the said commission occurring by resignation or otherwise shall be filled by such justice, and the persons appointed to fill such vacancies shall be appointed for the unexpired term only; such persons so appointed, when duly qualified, constituting such commission and their successors are hereby created a body politic, with

Judge of Supreme Court to appoint commission.

Official name.

Terms.

Vacancies.

power to sue and be sued, to use a common seal and make by-laws; the members of any such commission shall serve without compensation, except that the necessary expenses of each commissioner for actual attendance on meetings of said commission shall be allowed and paid. No persons employed by the said commission shall be a member thereof; before entering upon the duties of his office each commissioner shall take and subscribe an oath or affirmation before the clerk of the county in and for which he is appointed to faithfully and impartially perform the duties of his office, which oath or affirmation shall be filed with the clerk of the county wherein the commission of which he is a member is appointed; every such commission shall annually choose from among its members a president and treasurer, and appoint a clerk or secretary and such other officers and employes as it may deem necessary to carry out the purposes of this act; it may also determine the duties and compensation of such employes, and make all rules and regulations respecting the same. It shall be the duty of the board of chosen freeholders in each county to provide such commission with a suitable office where its maps, plans, documents, records, and accounts shall be kept, subject to public inspection at such times and under such reasonable regulations as the commission may determine.

Expenses paid. Oath. Organization. Office.

"2. The director of the State Experiment Station shall be a member ex officio of each commission and shall co-operate with them for the effective carrying out of their plans and work. The said director shall serve without compensation, except that the necessary expenses actually incurred by him in the attendance on meetings of said commissions shall be allowed and paid. He shall furnish the said commissions with such surveys, maps, information, and advice as they may require for the prosecution of their work, or, as in his opinion, will be of advantage in connection therewith.

Duty of director of experiment station.

"3. Every such commission shall have the power to eliminate all breeding places of mosquitoes within the county wherein it is appointed, and to do and perform all acts and to carry out all plans which in their opinion and judgment may be necessary or proper for the elimination of breeding places of mosquitoes, or which will tend to exterminate mosquitoes within said county.

Powers.

"4. Said commission shall, on or before the first day of April in each and every year, file with the director of the State Experiment Station a detailed estimate of the moneys required for the ensuing year, and a plan of the work to be done and the methods to be employed. The said director shall have the power to approve, modify or alter the said estimates, plans and methods, and the estimate, plan and method finally approved by him shall be by him forwarded to the board of chosen freeholders in each county on or before the first day of May following its receipt.

Annual estimate.

"5. It shall be the duty of the board of chosen freeholders of each county, or other body having control of the finances thereof, to include the amount of money approved by the director of the State Experiment Station, annually in the tax levy; *provided, however,* that in no year shall the amount so raised exceed the amount hereinafter specified, to wit, in counties where the assessed valuations are not more than twenty-five million dollars, a sum not greater than one mill on every dollar of assessed valuations; in counties where the assessed valuations are not more than fifty million dollars a sum not more than one-half of one mill on every dollar of assessed valuations; in counties in which the assessed valuations are in excess of fifty million dollars a sum not more than one-quarter of one mill on every dollar of assessed valuations. **Appropriation.** **Proviso; amount raised annually.**

"6. The moneys so raised, or so much thereof as may be required, shall be paid from time to time to the said mosquito commission on the requisition of said commission, duly signed and approved by the president and secretary thereof. **Payments.**

"7. It shall be the duty of each commission annually, on or before the first day of November in each year, to submit to the director of the State Experiment Station and to the board of chosen freeholders in their respective counties a report setting forth the amount of moneys expended during the previous year, the methods employed, the work accomplished and any other information which in their judgment may seem pertinent. **Annual report.**

"8. Nothing in this act shall be construed to alter, amend, modify or repeal the provisions of chapter 134 of the laws of 1906, or alter, amend, modify or repeal any act now existing conferring upon State or local boards of health any powers or duties in connection with the extermination of mosquitoes in said State, but shall be construed to be supplementary thereto. **Act considered supplementary.**

"9. This act shall take effect immediately.

"Approved March 21, 1912."

The Essex County, New Jersey, Mosquito Extermination Commission was organized in March, 1912, and under this law obtained \$75 000 from the Board of County Freeholders to carry out the work in the county during 12 months ending May 1st, 1913. There are about 600 000 people in Essex County, and this tax amounts to 12½ cents per inhabitant per annum. The work accomplished during 1912 has been extraordinary; on every side the same story is heard: "We have never known a year when we have seen so few mosquitoes. We might almost say we have hardly seen one in our house this year."

The organization of this Commission is as follows:

1.—Six Commissioners serving without salary.

2.—One Chief Inspector, salary, $1 800 per annum.

3.—One Secretary, salary, $1 200 per annum.

4.—One Assistant Chief Inspector, $1 500 per annum.

5.—Three Deputy Chief Inspectors (who must qualify as engineers), salary, $1 200 per annum.

6.—Forty-five Inspectors at $3 per day.

7.—Thirty to forty laborers at $2 per day.

8.—Stenographer and Clerk, salary, $600 per annum.

Each inspector patrols a distinct district, making his rounds once in every 10 days. In certain sections of the City of Newark, a day's work for an inspector is from 100 to 300 houses; a day's work for an inspector in the outlying districts is far less. All breeding places which cannot be eliminated at once are recorded on maps made by the Deputy Chief Inspectors.

In closing, the writer wishes to emphasize:

1.—That all standing water does not breed mosquitoes, and great economy can be effected in the cost of drainage and oiling if each pool of standing water is carefully examined for mosquito larvæ before any work is expended thereon.

2.—That we will never be relieved of the work of mosquito prevention. The fight will be continuous, and any slackening of vigilance on the part of the organization will invite the immediate reappearance of the pest.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

THE SANITATION OF CONSTRUCTION CAMPS.

By Harold Farnsworth Gray, Jun. Am. Soc. C. E.

To be Presented December 18th, 1912.

A problem which often confronts the engineer or contractor engaged in work which requires the establishment of a camp, is the keeping of labor steadily on the job in such physical and mental condition that each man puts forth his best efforts. This is especially true when work is prosecuted in new and unpopulated country, where life at its best must be rough and at times hard. The engineer in charge may have to carry on the work with a labor force depleted by sickness. As a result, many may leave permanently, and many of those who remain may be in such poor health that their physical efficiency is greatly impaired. A shifting or half-sick labor force cannot be efficient.

The writer has seen camps where a considerable proportion of the men were laid off because of intestinal disorders, due largely to poor sanitation, and where the efficiency of the force was reduced perhaps one-half. He has been in camps in malarial regions where the reduction in efficiency of the force was known to be at least one-quarter, and at times was probably greater. Such conditions represent a serious loss to the contractor, a loss which can be almost entirely eliminated by the observance of a few simple rules

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

based on the present knowledge of disease prevention. In the following the writer proposes to set forth briefly some of the principles of sanitation which are applicable to camp conditions, in the hope that he may convince the Engineering Profession that good camp sanitation is an economy, and not an expense.

Camp Location.—The matters of site and soil conditions are not of prime importance. Even under the poorest natural conditions, good health may be maintained in the camp if the proper sanitary measures are taken, but the better the natural conditions, the fewer and less costly will be the sanitary measures required.

Preference should be given to an even surface with a slight natural slope. An even surface promotes convenience and accessibility, and a slight slope gives a quick run-off for possible rain water. In cool climates a sunny exposure should be selected, and, in hot climates, the shady side of a valley, if possible. Low places, swamps, and wet areas should be avoided, as these may be breeding places of malaria-bearing mosquitoes. A light, porous soil is an advantage, not that disease lurks in the emanations from damp ground, but because of the added comfort of dry quarters.

Housing.—The form of housing will depend largely on the duration of operations and on transportation facilities. Whether tents, portable houses, or the more permanent wooden bunk-houses are used, ample provision for the free entrance of air and sunlight should be made. If wooden bunk-houses are used, the window space should be ample, and the windows swung inward on hinges at the side of the frames. In warm climates, in summer, and especially in fly and mosquito regions, the doors and windows should be screened, as described later.

One or more able-bodied men, as required, should be assigned to janitor duty. Each day all quarters should be opened as widely as possible for several hours during the morning for ventilation, and thoroughly cleaned by the janitor. Dirty conditions and violations of the camp rules should be reported by him, and the offender warned, disciplined, or discharged, according to the magnitude and frequency of the offense. With proper cleanliness, disinfectants are not required, except in unusual circumstances.

In cold climates, and during the winter, stoves (of a type which

carries the products of combustion out of the room) should be provided, and fuel supplied to the men.

Mess-House.—The mess-house and kitchen should be especially well ventilated, but all doors, windows, and other openings should be screened with a copper or bronze wire of No. 18 mesh screen. Doors should be provided with springs to keep them closed. In order to prevent the entrance of insects around the edges of screens, the frames should close flush against wood strips nailed to the casing.

All reasonable means for obtaining cleanliness in the preparation and handling of food should be provided, and the kitchen and mess-house should be thoroughly cleaned after each meal. It is desirable that the various sources of food supply be known, where possible, and only such foods accepted as are wholesome and are produced and marketed under reasonably clean conditions. This is no doubt difficult under average conditions, but is worth considerable effort. As far as possible, the diet should be suitable to the season. No person afflicted with a communicable disease should be allowed to work in any capacity in the kitchen or mess-house.

Garbage.—Metal receptacles, provided with tight metal covers, should be supplied and used for the kitchen wastes, both solid (garbage) and liquid (slops). In camps which are to be maintained for a period of two weeks or longer, all garbage and slops should be destroyed by incineration, as any other form of disposal will probably permit fly-breeding. In no case should either garbage or slops be emptied on the ground, even at a considerable distance from the camp. Fig. 1 shows a simple and inexpensive garbage and slops incinerator, designed by Maj. Paul F. Straub, G. S., U. S. Army, and is given here by his permission. This incinerator was used successfully at the maneuver camp at San Antonio, Tex., in 1911. With proper attention it will destroy 100 gal. of slops and 23 cu. ft. of garbage in 12 hours, with a fuel consumption of $\frac{1}{16}$ cord of wood. With the ordinary attention which would be given in a construction camp, lacking in rigid military discipline, the fuel consumption would be greater, but $\frac{1}{8}$ cord per day should be ample for the destruction of all garbage and slops created in the preparation of food for 100 men.

The liquid slops are evaporated by being poured slowly along the walls of the incinerator, and the garbage is placed on top of the fuel. If rock for the construction of the incinerator is not avail-

able, a trench of the same form and capacity may be dug in the ground, and operated in the same manner. Great care should be taken to see that the garbage is thoroughly consumed; otherwise, fly-breeding may ensue if the incinerator is not used constantly.

Latrines.—Indiscriminate defecation and urination in the vicinity of the camp should be prohibited, and a sufficient number of latrines

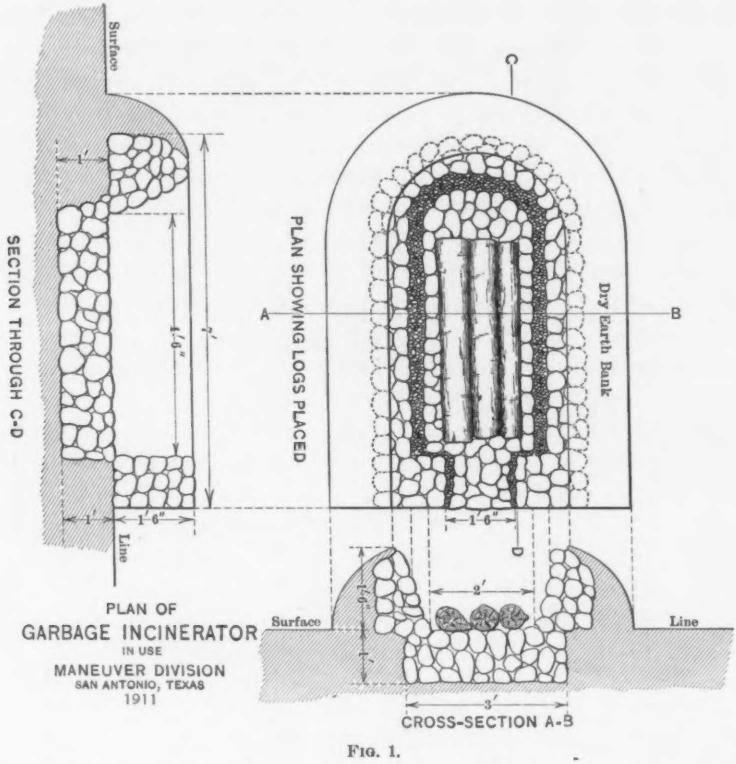
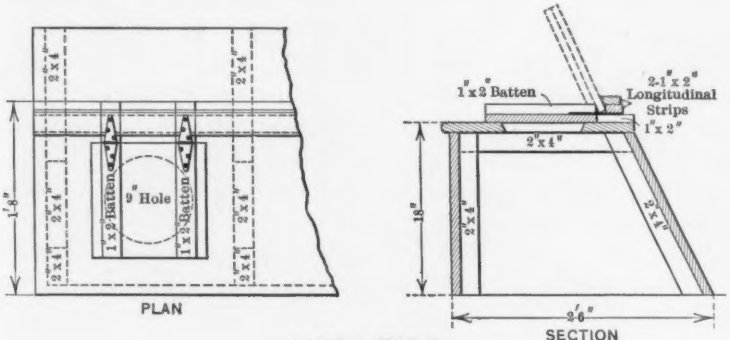


FIG. 1.

constructed at convenient places. A two-hole seat with a urinal trough is a good standard for the average camp, and the latrines should be placed so that the maximum distance from any house or tent to the nearest latrine does not exceed 100 ft. With easy accessibility of latrines, promiscuous defecation is more easily discouraged.

The latrines consist of three parts: the pit, the box, and the shelter. The pit should be 6 ft. or more in depth, and in section

slightly smaller than the box. The box is shown by Fig. 2 (also by permission of Maj. Straub). It should be made of tongued-and-grooved boards, as ordinary lumber tends to warp and shrink, so that it is difficult to keep the box fly-tight, reliance being placed on the tightness of the box to prevent the access of flies to the contents of the latrine. The covers of the holes should drop back automatically into place over the holes when the seat is not occupied. In summer no roof is provided for the shelter which, for privacy, is built around the latrine, as the direct action of sunlight and air is desirable as a disinfectant and deodorant. During the winter a roof may be provided, and during rains, at other seasons of the year, a tarpaulin should be used.



CAMP LATRINE

FIG. 2.

A metal urinal trough may be attached to the side of the latrine shelter, discharging into the pit through the box *via* a trap. The pit should be disinfected daily by being burned out with oil or oil-soaked straw. The urinal trough should be flushed with water and limed, and the interior of the box and the pit should be limed after being burned, to deodorize the latrine. Cleanliness in the use of latrines must be strongly insisted on, especially if the laborers are of foreign nationality.

Stables.—The stable, especially the stalls, should be cleaned out thoroughly each day, and the manure removed to a point at least ½ mile from the camp and burned with oil. In a hot, dry climate, it will suffice to remove the manure to this distance and spread it out thinly on the ground; either method will prevent fly-breeding.

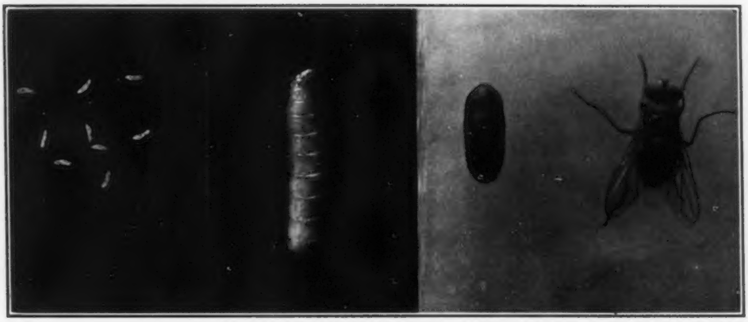
Under average conditions, horse manures are the favorite breeding places of flies. From estimates based on actual counts of representative samples, it has been shown that a manure pile weighing 1 000 lb. may contain upward of 450 000 fly maggots at the end of 4 days' exposure. Therefore, unusual care should be taken of manures, and the stables kept clean. There is no substitute for cleanliness; treating the stalls and manure with oils, chemicals, and poisons is in general more expensive and less effective than cleanliness and the proper disposal of manure.

Water Supply.—An endeavor should be made to obtain a pure water supply. Where the water is known to be polluted, it can be sterilized by calcium hypochlorite (chloride of lime or bleaching powder), used at the rate of from 10 to 40 lb. per million gallons, depending on the intensity of pollution. When used in this amount no taste or odor of the chemical will be noticed, and the water will be potable and practically sterile. For more permanent camps with polluted water supplies, sand filters can be constructed cheaply of wood, and can be made to give a fairly good effluent. Wells should be protected from surface contamination, and should not be placed in the vicinity of latrines.

Where camps are located on the water-shed of a city water supply, it will be necessary to take precautions to prevent the pollution of the supply by the camp. All such necessary precautions are usually covered by the laws of the State in which the camp is located.

Baths and Laundry.—In many respects personal cleanliness is one of the most important factors in health preservation, for the reason that disease is largely transmitted by contact, from person to person. It is advisable, therefore, that facilities for keeping personally clean be provided, and the men encouraged to make use of them. Shower baths can be easily and cheaply constructed. A simple and effective shower can be made from a 2-ft. length of 1-in. pipe, capped; along the lower side of the pipe are bored three parallel rows of $\frac{1}{16}$-in. holes set $\frac{3}{8}$ in. apart. The pipe projects horizontally from a tee in the vertical supply pipe, at about 8 ft. above the floor. The latter should be of wood with open joints. The bath should be provided with a shelter, and be without a roof in summer.

Laundry equipment and supplies, such as tubs, washboards, and soap, should be furnished gratis.



(*a*) (*b*) (*c*) (*d*)

FIG. 1.—DEVELOPMENT OF THE COMMON HOUSE FLY.
(*a*) EGGS. (*b*) LARVA OR MAGGOT. (*c*) PUPA. (*d*) ADULT FLY.

FIG. 2.—KNAPSACK SPRAY-PUMP IN USE, SPRAYING OIL ON A TYPICAL ANOPHELINE BREEDING PLACE.

Rubbish.—Unless cared for, considerable rubbish tends to accumulate in any camp. It should be collected daily by the janitor, removed to a specified place, and burned. Everything which tends toward neatness and cleanliness should be encouraged.

Flies.—The principal breeding places of flies, namely, manure piles, garbage heaps, and latrine pits, have been mentioned. They may also breed in any collection of decomposing organic matter, no matter how small. The fundamental theorem of fly control is cleanliness combined with proper destruction of waste products. Where destruction is not feasible, the rendering of such wastes unsuitable to fly-breeding, or inaccessible to flies, may be substituted.

Under midsummer conditions, flies will develop from egg to adult in from 10 to 14 days. The adult female lays from 75 to 125 eggs on the surface of the breeding material, and there may be several such layings during the average life of the insect. The eggs hatch in from 12 to 24 hours; the larvæ or maggots feed from 4 to 7 days, crawl away to the lower part of the breeding material, or into the adjacent earth, and pupate; in the pupal stage they remain quiet 4 days or more, and emerge full-grown flies. Fig. 1, Plate CXXX, given by courtesy of William B. Herms, Assistant Professor of Applied Parasitology, University of California, shows the four stages in the development of the common house-fly.

The rapidity of development is dependent on temperature, being accelerated by warmth and retarded by cold. Owing to the fairly constant warmth of decaying organic matter (for example, manure), the larval period is very nearly constant. The pupal period, being spent away from the decaying material, shows the greatest variation, and, in winter, may be lengthened to several months.

Owing to the fact that the fly maggot is very resistant to poisons and insecticides, it will be found cheaper and more effective, as previously stated, to destroy or remove the breeding material, than attempt to treat accumulations with poisons.

It has been proven conclusively that under conditions such as are usual in camps, flies may be gross transmitters of such diseases as typhoid, dysentery, and various intestinal disorders. During the Spanish-American War, when more soldiers were killed by disease than by bullets, large numbers of flies were often observed on the food in the mess-tents, the bodies and feet of many of them being

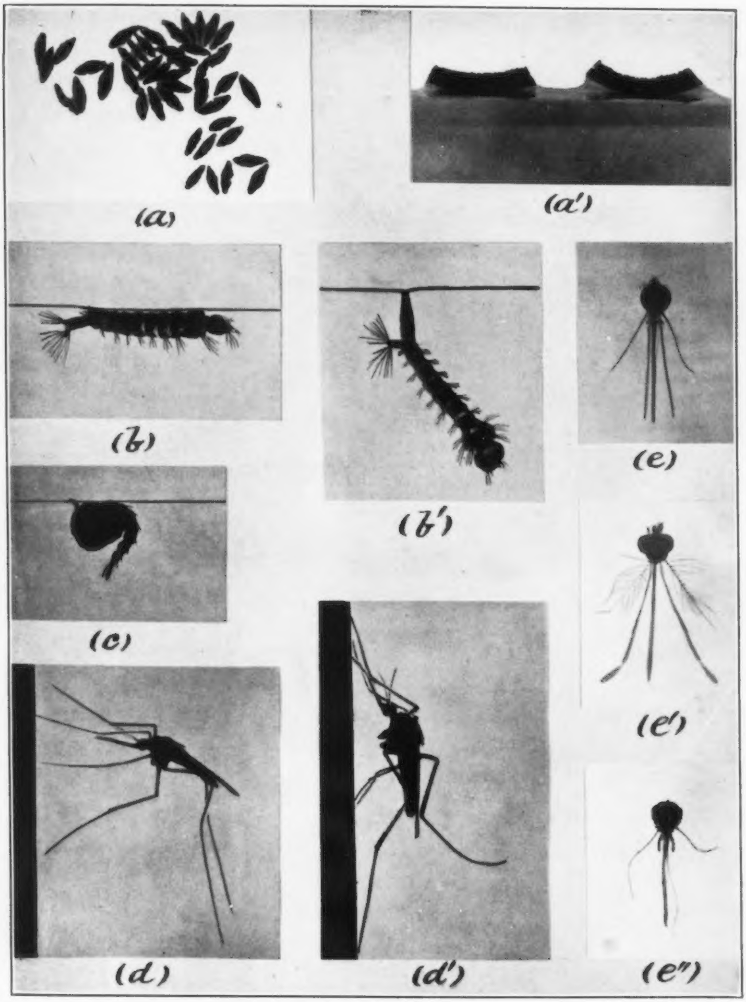
whitened by the lime in the latrines which they had recently visited.

It may be stated almost as an axiom that the sanitary condition of a camp varies inversely as the numbers of flies present.

Malaria and Mosquitoes.—It is now conclusively proven that mosquitoes of a certain type are the sole transmitters of malaria. This disease is most prevalent during the warm months of the year, when the largest amount of construction work is carried on. In malarial regions the lost efficiency of the labor force from this disease is often 25% and more. Even where malaria-bearing mosquitoes are not present, and those of the common type are numerous, it is advisable to take measures for their destruction, as a night disturbed by these insects does not put a man in the best condition for a hard day's work. As previously mentioned, the doors and windows of wood houses should be screened and, in mosquito districts, if tents are used, bed-nets supported by frames should be supplied.

The two types of mosquitoes, the *Anophelines,* or malaria transmitters, and the *Culicines,* or common mosquitoes, are shown in various stages of their life history on Plate CXXXI. The differences in the two types are easily recognized in three of the four stages. The *Culicines* lay their eggs on the water surface in boat-shaped masses of from 250 to 750 eggs; the *Anophelines* lay their eggs singly or in irregular clumps. In the larval or wriggler stage, passed in water, the *Culicines* project a posterior breathing tube to the surface, the remainder of the body hanging head downward at an angle with the surface; the *Anophelines* breathe while lying parallel to, and with every segment touching, the surface. The differences in the pupal or tumbler stage are not sufficiently well marked to be easily distinguished by the unaided eye. In the adult stage, two methods may be used to distinguish the types. The resting attitudes are different; an *Anopheline* resting on a plane surface holds the posterior part of the body away from the surface, the line of the body thus making a distinct angle of from 25 to 55° with the surface, whereas the *Culicine* adult rests with the line of the body parallel to the surface, or in a humpbacked position. The best method of distinction in the adult stage is based on the mouth parts, which can be seen with the unaided eye, but can be observed better with a low-power magnifying lens. In the *Anopheline* females, the palpi (the two appendages im-

PLATE CXXXI.
PAPERS, AM. SOC. C. E.
NOVEMBER, 1912.
GRAY ON
SANITATION OF CONSTRUCTION CAMPS.



DEVELOPMENT OF THE MOSQUITO.

(a) ANOPHELINE EGGS (AFTER HOWARD). (a') CULICINE EGG-BOATS.
(b) ANOPHELINE LARVA. (b') CULICINE LARVA OR WRIGGLER.
(c) ANOPHELINE PUPA OR TUMBLER.
(d) RESTING ATTITUDE OF ANOPHELINE ADULT.
(d') RESTING ATTITUDE OF CULICINE ADULT.
(e) HEAD AND APPENDAGES OF FEMALE ANOPHELINE.
(e') HEAD OF MALE MOSQUITO, EITHER TYPE.
(e") HEAD AND APPENDAGES OF CULICINE FEMALE.

mediately on each side of the central proboscis or beak) are approximately as long as the proboscis, while in the *Culicines* the palpi are shorter than half the length of the proboscis. The males, which do not suck blood, can be distinguished, if found, by the feathered or plumose antennæ or feelers.

Mosquitoes breed in stagnant or quiet water, the *Anophelines* preferring clear, fresh water standing but a few inches on grassy land. The *Culicines* are not as fastidious in their choice of breeding places, often thriving in the foulest water. The first principle of mosquito control, therefore, is the removal of all stagnant water. This may be effected by drainage, by filling in small low spots which cannot be readily drained, by emptying vessels which contain useless water, and by screening all others. Where drainage or filling in is not practicable, as will usually be the case in construction camps, oil or chemicals may be used to destroy the mosquito larvæ.

Oil sprayed on the surface of water which is breeding mosquitoes will kill them effectively by shutting off the air supply, the larvæ and pupæ dying by suffocation. The oil should have a specific gravity of about 30° Baumé, and may be applied conveniently and economically by a knapsack spray-pump, as shown in Fig. 2, Plate CXXX. Small ponds may be treated by throwing in several handfuls of cotton waste soaked in oil, which will gradually feed a film of oil on the surface for some time, and may be renewed at intervals.

Under midsummer conditions, the development of the mosquito requires from 10 to 14 days; in the spring and autumn, this period is somewhat longer. To prevent breeding, therefore, it is necessary to apply the oil by spraying once every 2 weeks in summer, and about once every 3 weeks in the spring and fall.

Where the water is full of vegetation, it is advisable to treat it first with copper sulphate (1 to 50 000) and then with oil. A preliminary copper sulphate treatment will be found an advantage where poisons or larvacides are used. Several poisons, such as nicotine sulphate, phinotas oil, and emulsions of a modified crude carbolic acid, have been used for mosquito destruction with varying success, but on the scale of operations which would be usually attempted for the protection of a construction camp, oil will be found to be cheaper and more effective.

It will seldom be necessary to carry on anti-mosquito operations

beyond a distance of 400 yd. from the camp, and not all the water in this territory will be found to be breeding mosquitoes. Only water which is actually breeding them need be treated.

Medical Treatment.—While the subject of medical treatment of the sick does not properly come under the provisions of camp sanitation, it is advisable to consider it briefly here, on account of its relation to the control of disease. The character of such treatment will depend on the size of the camp and the country in which the work is carried on. For small camps, in sparsely inhabited regions, a medicine chest of standard remedies, with bandages, liniment, and first aid to the injured materials, should be provided, to be administered gratis as needed. In malarial regions, in addition to screening and mosquito destruction, a daily prophylactic dose of quinine should be urged on the employees. Where small camps are within easy reach of a town, arrangements should be made with a physician to call at regular times, and as needed in emergencies, and, in addition, the medical and first aid equipment should be provided for minor complaints.

Where several camps are working a large number of men, directed from a central field office, it is advisable, and usually practicable, to install a field hospital with a resident physician, who, in addition to his hospital duties, makes periodic inspections of the camps. The expense of the hospital is usually provided for by nominal deductions from the pay of each employee, which entitles him to free medical attendance if sick or injured. This is often done on works of magnitude, an example with which the writer is familiar being the Los Angeles Aqueduct.

In any case, an employee suffering from any form of communicable disease should at once be laid off and isolated from the others, to prevent as far as possible the further spread of the disease.

The Value of Good Sanitation.—In its broadest sense, sanitation includes all methods and procedures necessary for the prevention of the spread of diseases. In its practical workings, it is limited to certain fields, which are not very well defined, owing to the interrelations of various factors in disease prevention. The procedure in disease prevention may be arranged roughly in three main divisions: First, the control of the source of infection, practically always a person sick with the disease; second, the control of the carrier or

means of distribution of the disease from one person to another; and third, the protection of the well persons individually.

The first division falls largely in the domain of the physician, though the engineer, by installing works for the prompt removal and purification of the infected discharges of the patient, plays his part here. The control of the carrier of infection, such as food, water, sewage, insects, etc., comprises the field of sanitation as usually defined, and in this field the engineer, chemist, bacteriologist, entomologist, and others, play the chief part. Personal hygiene largely covers the third division. All three divisions are links in a chain which is no stronger than its weakest link. Under camp conditions, however, it is not practicable to take the measures which are essential in an urban community, and it is not possible to pay much attention to personal hygiene. The bulk of disease prevention, therefore, must fall in the second division.

There are few engineers who are not familiar with the usual conditions in camps. Foul-smelling, open latrines swarming with flies, dirty and poorly ventilated quarters for the men, vile food, filthy mess-tent, and swarms of flies on and in everything, are the average conditions. Such being the case, it is well-nigh impossible for any man to remain in good physical condition, and it is the laborer's physical condition which largely determines the amount and character of his output. If a part of the labor force is weakened by ill-health, just that much less than the maximum possible work can be obtained. The contractor expects the maximum for his greatest profits; if he gets less than the maximum, he sustains a loss. A reduction of 25% in the efficiency of the labor force is not unusual. If we consider a camp of 100 men at an average daily wage of $2, this daily loss amounts to $50, and in a 30-day month would reach a total of $1 500. The additional cost of good sanitation above the amount spent for the poor sanitation which is at the bottom of this loss, would probably not exceed $250 per month, leaving a net profit, chargeable to good sanitation, of $1 250. The average saving would undoubtedly be less than this amount, but the writer does not believe that he has based this estimate on an extreme case. However, be the saving much or little, good sanitation in a camp is emphatically an economy, not an expense. Aside from ignorance, perhaps the chief

reason that camp conditions are usually so bad is that the contractor can actually see in dollars and cents the money expended for good sanitation, while the saving cannot be expressed so accurately or in such tangible form. Nevertheless, the economy can be shown at least approximately in every case, and in extreme cases may represent the difference between a profit and a loss on the contract.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

STREET SPRINKLING IN ST. PAUL, MINN.

Discussion.*

By A. H. Blanchard, M. Am. Soc. C. E.

A. H. Blanchard, M. Am. Soc. C. E.—This paper brings to the attention of municipal engineers the value of cost data and accurate records of construction details. The speaker, however, only refers to this feature of the paper in order to express his appreciation of the author's services in bringing before the Society a comprehensive plan covering cost data and records in a field in which very little work of this character has been done. Mr. Blanchard.

A paper on the broad problems of the economics of street watering seems to be opportune. Up to this time there has been comparatively little discussion relative to the efficiency of street watering. In considering the problem from this standpoint, a question which occurs to any one interested in the subject is: What are the fundamental reasons for sprinkling streets with water? There are engineers who claim that the reason is mainly to lay the dust. Street sprinkling, as done by these engineers, is used for that purpose not only on macadam roads, but on pavements of all types. If pavements are properly cleaned (by methods which are adaptable in practically all American cities), is there any necessity to sprinkle them to lay the dust? The answer is in the negative, because, with a proper system of street cleaning, dust should not exist in such quantities as to require any process of dust laying. The question of laying dust on macadam roads naturally brings up for consideration the relative economics by watering and by other methods which have come into use within the past decade. This feature will not be considered by the speaker, as it has formed the subject of many discussions before this Society.†

*Continued from October, 1912, *Proceedings*.

† *Transactions*, Am. Soc. C. E., Vol. LXV, pages 462 to 466, and Vol. LXXIII, pages 33 to 43.

Mr. Blanchard.

Another reason advanced by some municipal engineers is that street watering materially cools the atmosphere. These engineers, however, use street sprinkling methods from the middle of April to the middle of October. It is doubtful if, in the majority of cities, engineers could state conscientiously that it is necessary to cool the atmosphere during two-thirds of the time when streets are watered. It would be of material value if a series of experiments should be undertaken to determine how much the atmosphere is cooled within a certain reasonable distance above the pavement and on adjoining property by street sprinkling with water. G. A. Soper, M. Am. Soc. C. E., who has spent considerable time in investigating the subject, is particularly positive in the statement that no cooling effect is appreciable. Although it is admitted by many that it may not be appreciable from the standpoint of the actual temperature, it is believed, however, that on macadam roads, under certain local conditions, there is a cooling effect—at least mentally. One excellent example of this effect is seen in the method used in Monte Carlo. Although the macadam roads of the Principality of Monaco are rendered dustless in the main by the use of bituminous materials, in the instance of the light-colored limestone macadam roads in the vicinity of the Casino, surrounding the beautiful gardens laid out in front of that magnificent yellow limestone building, the streets are watered not only to give the effect of coolness, but also, from the esthetic standpoint, in order that the limestone macadam thus treated will harmonize with the beautiful buildings and residences which border these boulevards.

A third reason which has been advanced by some engineers is that street watering serves to clean the streets. The speaker does not believe that it is a question for consideration whether street sprinkling properly conducted does or does not clean the streets. Certainly, it is usually impracticable to clean macadam roads by watering without the creation of mud or the disintegration of the surface. With the figures at hand, it certainly is not economical to clean pavements by street sprinkling when they are used under conditions for which they are suitable and economical.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

A WESTERN TYPE OF MOVABLE WEIR DAM.

Discussion.*

By Thomas C. Atwood, M. Am. Soc. C. E.

Thomas C. Atwood, M. Am. Soc. C. E.—The author has performed a distinct service by bringing before the Society the subject of inexpensive movable dams for use in irrigation canals, small streams, etc., and in pointing out correct principles of design. Mr. Atwood.

As he well states, "Their construction has largely been a growth of local or individual custom and experience, rather than of engineering design, and in many cases this is painfully apparent." It is to be hoped that the recent agitation for a stricter supervision of dams, following the failure of that at Austin, Pa., may extend even to such small structures as these, and that all of them may be placed in the hands of competent engineers.

This type of dam seems to be better adapted to irrigation canals than to streams, due to the likelihood of driftwood in the latter lodging against the bents in time of flood and holding back the water, in spite of the removal of the flash-boards, and, perhaps, causing the loss of the structure. For such locations, especially where the dam is a high one, as the diverting dam of the San Joaquin and Kings River Canal and Irrigation Company across the San Joaquin River, where the height is given as 16 ft., a type of movable dam which gives a clear waterway when lowered is usually to be preferred. Many such dams have been built by the United States Army Engineers, and have proved very successful. Although permanent in character, the cost of the waterway is not excessive, the principal expense being in the provision for navigation. The cost of the navigable pass of

*This discussion (of the paper by W. C. Hammatt, M. Am. Soc. C. E., published in May, 1912, *Proceedings*, and presented at the meeting of September 18th, 1912), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Atwood. Dam No. 26, on the Ohio River, is given by the U. S. Engineers' office as about $175 per ft., the dam being built on a rock foundation, and for a maximum head of 14.2 ft. Mr. J. W. Arras, U. S. Assistant Engineer, gives the probable cost of a Chanoine dam on piles in a gravel foundation as from $250 to $300 per ft. for a structure about 16 ft. in height, according to local conditions.

The failure of Dam No. 26 was due apparently to the failure of the rock foundation, although the unwatering of the site may reveal some other cause. This is worthy of note, in view of the almost uniform success of these dams when founded on the river gravel.

The author does well to lay stress on the importance of the foundation, and his suggested design, as shown in Fig. 5, is excellent in this respect on most points.

The subject of scour does not seem to be adequately treated, however, although Mr. Hammatt called attention to this point in his discussion* on the Yuma River Débris Barrier, where he suggested an unattached apron of large concrete blocks which would follow the bottom down, as scour takes place below the dam, and prevent the gravel under the dam itself from being washed away. This method is excellent, and has been used successfully, as have a number of others, as described by the writer in discussing the paper on the Yuma River Débris Barrier mentioned above.

The suggested design, as given in Fig. 5, is weak in this respect, as scour may reasonably be expected when the flash-boards are raised and the direct current strikes the small bulkhead, *H,* and jumps over it. This can be helped by moving this bulkhead nearer to the main dam, extending the plank apron farther down stream, or placing rip-rap below the apron to prevent scour and follow the bottom down if any occurs.

The best method, however, and probably the cheapest, is to slope the apron upward in the direction of the flow, with perhaps 2 ft. rise in 20 ft., giving the same depth of water just below the dam, but doing away with the bulkhead, *H*. There will still be scour, but it will be transferred to such a distance below the dam that the foundation of the structure will not be endangered, the tendency of the eddy being to bring gravel back toward the apron, rather than to scour that immediately adjacent.

* *Transactions*, Am. Soc. C. E., Vol. LXXI, p. 229.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

THE SIXTH AVENUE SUBWAY OF THE HUDSON AND MANHATTAN RAILROAD.

Discussion.*

By Messrs. T. B. Whitney, Jr., William J. Boucher, Lazarus White, and H. L. Oestreich.

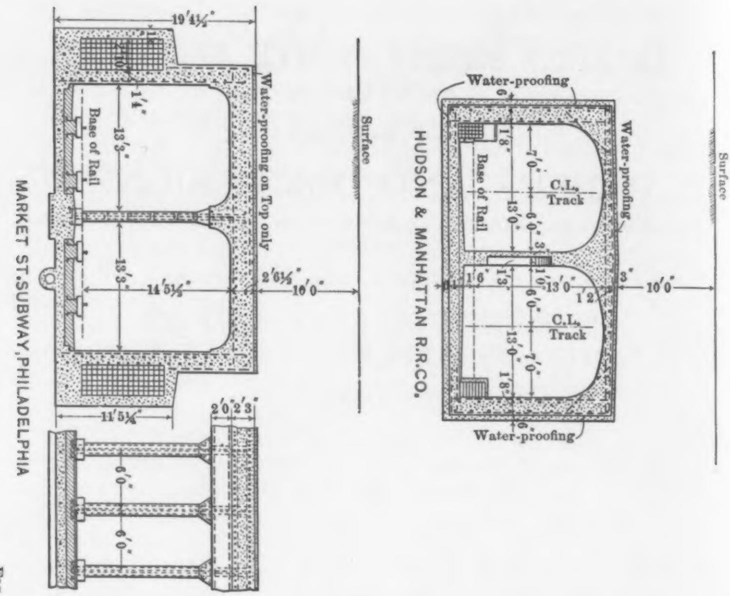
T. B. Whitney, Jr., M. Am. Soc. C. E. (by letter).—Judging from the author's statement that the weight of the reinforcing steel per cubic foot of concrete is excessive, it might be inferred that some abnormal percentage of steel was used in the design of the reinforced concrete in this structure. However, the steel ratio is not more than three-fourths of 1%, except in the side-walls which required reinforcing in both faces, where it amounts to 1.20 per cent. The longitudinal bars used to prevent shrinkage cracks averaged about fifteen-hundredths of 1 per cent. A minimum spacing of 6 in. from center to center was used for the steel in order to facilitate the pouring of the concrete. Mr. Whitney.

The arched roof formed a characteristic feature throughout the underground structures of the Hudson and Manhattan Railroad, and this led to its consideration in the design of the subway section on Sixth Avenue. The adopted section consists of a double, arched roof supported by a center wall and side-walls. The side-walls are designed as reinforced concrete slabs to carry the earth pressure as well as the thrust of the arches, and are tied together at top and bottom by tie-bars in the roof and floor.

A comparison of this subway section with two other well-known subway sections shows that the quantities of excavation, steel, and

* This discussion (of the paper by H. G. Burrowes, M. Am. Soc. C. E., published in August, 1912, *Proceedings*, and presented at the meeting of October 2d, 1912), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Whitney.



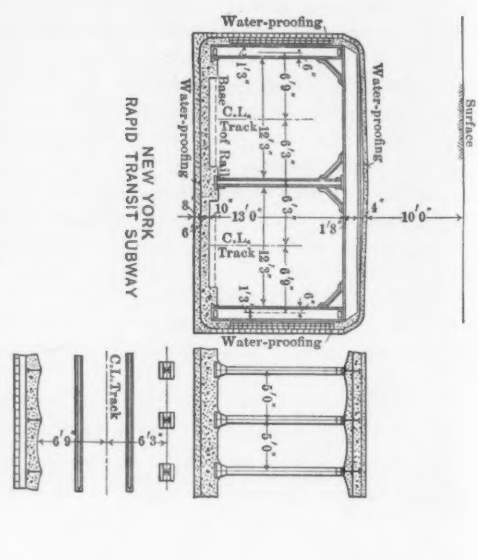


FIG. 4.

QUANTITIES PER LINEAR FOOT OF TWO-TRACK SUBWAY

	Inside Clearance	Outside Width over all	Excavation in Cubic Yards	Concrete in Cubic Yards	Reinforcing Steel in Pounds	Structural Steel in Pounds	Total Steel in Pounds	Water-proofing in Square Yards	Concrete over Water-proofing in Cubic Yards
Hudson & Manhattan	13'0"	31'7"	33.90	5.67	820	—	820	10.42	1.50
N. Y. R. T. Subway, f = 20,000 lb.	12'3"	29'10"	31.80	4.10	—	816	816	9.78	1.47
N. Y. R. T. Subway, f = 16,000 lb.	12'3"	29'10"	31.80	4.10	—	991	991	9.78	1.47
Market St. Subway, Philadelphia	13'3"	37'10"	43.70	7.80	574	206	780	3.50	0.29

Mr. Whitney.

concrete, are slightly in its favor, when the sections are reduced to the same dimensions, using the same loading and unit stresses in each case. The economic features may be compared by referring to Fig. 4. It will be noticed that the New York Subway has the smallest side clearance and that the steelwork is calculated on a basis of a unit stress of 20 000 lb., which accounts for the smaller quantities.

The permanent structure was designed to conform with the live and dead loads as given by the specifications of the New York Public Service Commission, though lower unit stresses were used than are prescribed by these specifications.

In each group of girders spanning the Pennsylvania Tunnel at 32d Street, the girder adjacent to the track carries half the trough track floor and train loads, in addition to its equal share of the

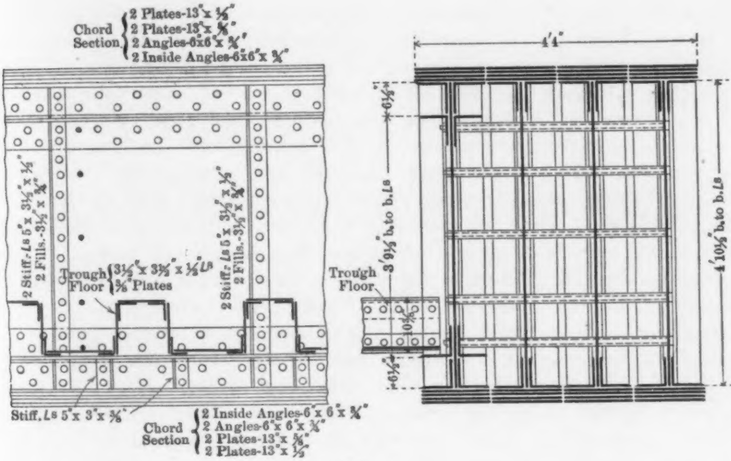


FIG. 5.

permanent overhead loads, consisting of station platform, columns, roof, 20 ft. of fill, street railway, street loads, and elevated railroad structure. Economy of space was essential, so the over-all dimensions of all the girders were kept the same, and the additional flange area required for the girders carrying the track floor and train loads was provided by extra sets of angles placed inside the main flanges of the girder. The increase in web thickness, to provide proper rivet spacing, gave a surplus of material sufficient to carry the additional shear, as shown by Fig. 5. Thus, the extremely heavy overhead loads are carried by the main body of the girder, while the comparatively light track floor and train loads are carried by the inner sets of flange angles.

Mr. Whitney. The trough track floor rests on the outstanding leg of the inner flange angles, which is braced by stiffener angles, 12 in. from center to center, these angles transmitting the loads to the web of the girder. This arrangement simplifies the renewal of these floors.

Mr. Boucher. WILLIAM J. BOUCHER, ASSOC. M. AM. SOC. C. E.—While in Chicago and associated with the organization planning the proposed subway system for that city, about 2 years ago, the matter of water-proofing was quite thoroughly investigated by the speaker. Having been connected with the construction of the New York Rapid Transit Railroad, he was well acquainted with the membranous method of water-proofing, but the integral method of mixing a powdered or liquid substance with the concrete had not been examined to any extent.

At least six manufacturers of compounds offered, and had tests made of, their products, among which were McCormick, Medusa, Truss-Con, Hercules, Ceresit, and Hydrolithic. Due to various causes, the tests were not in every way satisfactory, but it was learned that laboratory tests were of little or no value. No subway has yet been built in Chicago, but the street railway tunnel under the Chicago River at La Salle Street has now been in use about a year. The reinforced concrete approaches to this tunnel, and the concrete-lined steel tubes forming the river crossing are water-proofed solely by mixing McCormick compound with the concrete. The speaker has been informed by a representative of the company handling that brand, that the tunnel and its approaches at present are as dry as could be desired, the only places where water ever entered being at terminations of a day's work. These cracks and openings were closed by drilling holes through the concrete and forcing grout composed of water-proofed cement back of and into the mass. The speaker is further informed that since the completion of the street railway tunnel under the Chicago River at Washington Street, which was constructed of plain concrete, it has been treated with a thin mortar or grout of water-proofed cement, at various leaky places (as well as in the pump-room of the tunnel, which is on the shore, but below river level), and the results have been very satisfactory.

It will be interesting to learn whether the result of using a compound with the cement in the Hudson and Manhattan Railroad, has been satisfactory; whether cracks occurred and caulking was necessary, and whether the extra cost of the compound was justified, rather than the use of a richer cement mixture and more care in placing the concrete.

The first subway built in New York was a steel-beam structure, walls and roof. The first experimental piece of reinforced concrete subway was built from the Rapid Transit Commissioner's plans on Lenox Avenue, northward from 141st Street, a cross-section of which is shown on Fig. 6. Then followed an era, covering about 6 years, in

Mr. Boucher.

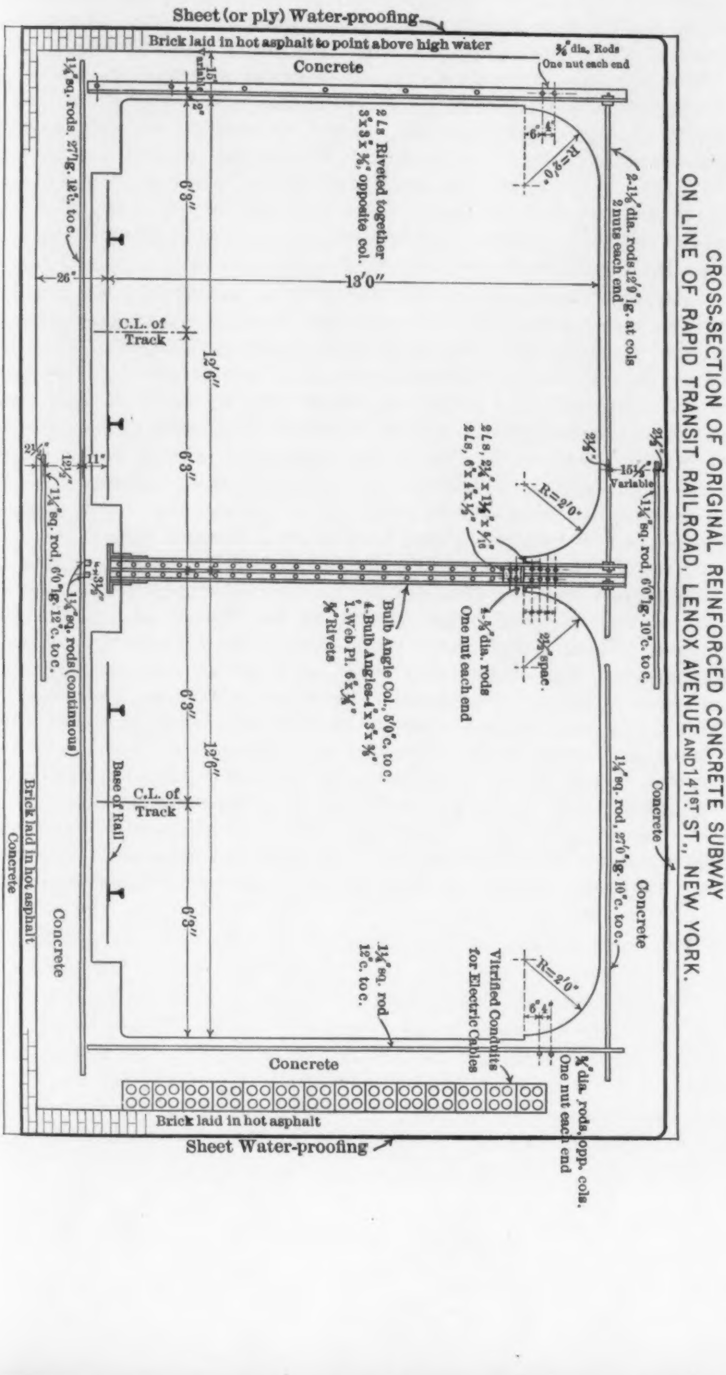


FIG. 6.

Mr. Boucher. which the subways were mainly of reinforced concrete, this construction being used under Battery Park, Manhattan, and Fulton Street, Flatbush Avenue, and Fourth Avenue to 40th Street, Brooklyn. The newer subways, however, are being designed and built as steel-beam structures, which seems to indicate that this type is preferable. Where surface cars and vehicle traffic, as well as elevated columns, must be supported, it seems to be far better to construct of beams, than rods, for then the loads may be transferred to the steelwork as soon as it is riveted, without waiting for the concrete to set. Can Mr. Burrowes state the reasons for adopting the reinforced type of construction for the Sixth Avenue subway?

Mr. White. LAZARUS WHITE, ASSOC. M. AM. SOC. C. E.—Would not Portland cement be a better compound than the material used on the subway work? In the work of the New York Board of Water Supply it was found that a little more Portland cement was better than any compound which could be added, no matter what its name—a mere name not producing dryness. If a wet concrete is used, and a liberal quantity of cement is carefully placed, that compound will be nearly water-proof; but nothing will make it water-proof at the joints. The advantage of the membrane method used on the subway is that it spans the joints and prevents water from getting through them.

Several tunnels were built for the Catskill Aqueduct, which, in the speaker's opinion, were dry enough to run subway trains through without any water-proofing other than the liberal use of Portland cement. These tunnels were deep, being several hundred feet below the ground, and after they were grouted off, successfully sustained very heavy ground-water pressures with very little leakage through the body of the concrete. A test* was made at the Wallkill Siphon, in which a stretch of tunnel was subjected to a head of several hundred feet of water. The arch had not been grouted, and leaked very slightly, but there was practically no leakage through the body of the concrete, showing conclusively that Portland cement is a sufficiently good water-proofing. The quantity of cement in this concrete, the speaker believes, was about 1.8 bbl. per cu. yd., corresponding to a 1 : 2 : 4 mix.

Mr. Oestreich. H. L. OESTREICH, M. AM. SOC. C. E.—The roof of the Fourth Avenue Subway, in Brooklyn, is of reinforced concrete, but no water-proofing was placed, a 6-in. layer of gravel on the roof acting as a drain to carry the water to the side of the tunnel. It was found that, if the structure leaked at all, it would be at the junction of two days' work. It is a question, therefore, whether Mr. Burrowes would not find the same conditions, even if he used the water-proofing powder in the concrete; in other words, while the slab placed one day might

* *Engineering News*, December 14th, 1911.

Mr. Oestreich.

be water-proof, the line at which it joined the next day's work would show a leak. That was the experience on the Brooklyn Subway. It seemed almost as though the concrete formed a better bond with any other material than with itself.

In order to prevent leakage from the roof at the junction of two days' work, a furrow about 1 in. deep and 2 in. wide was cut into the concrete along this line and filled with tar. This helped the process of silting up, and the roof became practically dry.

On two contracts of the Fourth Avenue Subway the brick in mastic was omitted on the sides and bottom of the subway, a 6-in. layer of 1:2:4 reinforced concrete being substituted. The reinforcing consisted of $\frac{3}{4}$-in., longitudinal rods occupying four-tenths of 1% of the area of the concrete, the intention being to cause the shrinkage to be in many and small cracks rather than in one large one. Overlapping the rods 48 diameters tended to strengthen the junction of two days' work. Constant pumping for 2 years has reduced the elevation of the ground-water temporarily, but it is probably too early to speak definitely of results.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

A BRIEF DESCRIPTION OF A MODERN STREET RAILWAY TRACK CONSTRUCTION.

Discussion.*

By Messrs. E. E. R. Tratman, Walter C. Howe, and Louis A. Mitchell.

E. E. R. Tratman, Assoc. M. Am. Soc. E. (by letter).—An interesting point suggested by this paper is the shape of the flangeway groove along the rail. It appears to be a rather wide, rectangular groove, having a flat bottom, vertical face, and sharp corner. It will be of interest to know whether this vertical face stands up under traffic, or whether the wheels of vehicles crush and crack it to an approximately beveled outline. The more usual method of forming the groove is to use nose-brick laid as headers with the top surface next the rail beveled so as to fit beneath the rail head. Still another plan is to use ordinary bricks (also laid as headers), tilted so that one end fits under the rail head while the top surface lies approximately in the contour of the crowning of the pavement between the rails. A third plan is to use a rectangular stretcher under the rail head (and projecting beyond it) and a higher bevel-edged stretcher level with the paving. In all these methods the groove or flangeway is of beveled or triangular section rather than rectangular. Mr. Tratman.

In the Springfield work, the crowning of the paving between the rails, as shown in Fig. 1, appears to be unduly high, interfering with the normal cross-section of the street, and making an irregular contour. This is especially the case as the paving is ½ in. below the rail head on the inside and ¼ in. below it on the outside, so that each rail forms a

*This discussion (of the paper by A. C. Polk, Assoc. M. Am. Soc. C. E., published in August, 1912, *Proceedings* and presented at the meeting of October 16th, 1912), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Tratman. distinct ridge in the pavement. This construction does not seem to be desirable in connection with a well-paved street surface, and it would be of interest to know whether the city authorities or the public have made any objection to it.

In regard to the wide spacing of the ties, experience has shown that the concrete immediately beneath the rail disintegrates or crushes, in some cases, due to the vibration and slight deflection of the rail, in spite of the fact that, theoretically, the rail has a continuous solid bearing.

Mr. Howe. WALTER C. HOWE, ASSOC. M. AM. SOC. C. E. (by letter).—This paper is both interesting and instructive. The writer has found very little published data on work of this class, although it is one of the most important features in the construction of modern pavements. Municipal engineers are often at a disadvantage in drawing up ordinances and specifications governing the control of paving in the excepted portion occupied by street railways.

The writer was Commissioner of Streets of Oakland, Cal., for a number of years, and observed that the initial point of failure of all asphalt streets was that portion immediately contiguous to the rails of the street railways, of both steam and electric roads. The asphalt surface adjoining the tracks was in a continual state of disrepair and disintegration. The failure of the pavement gradually extended outside of the railroad's portion and into that of the city. These conditions gradually became so serious that it was found necessary to adopt new and drastic ordinances governing the type of construction to be done by steam and traction lines occupying city streets. Few, if any, rails of modern type had been used by the companies previous to this action. A type of construction somewhat similar to that described by Mr. Polk was considered. The concrete extended to a depth of 6 in. under and completely across the length of the ties, but no steel reinforcement was used in the foundation. Strong opposition to this type of construction was made by the railway companies, and, as a compromise, rock ballast 6 in. deep was substituted for the 6 in. of concrete under the ties. There is no doubt that the class of construction shown by Mr. Polk is far superior, embodying, as it does, steel ties and a solid reinforced concrete foundation under them, thus affording extreme rigidity with a naturally decreased vibration. Its cost, however, is such as to preclude its use except in the case of street railway companies who are in excellent financial condition and whose officials are disposed and ready to meet with the municipality and adopt the most modern type of rail and foundation.

In many instances the official in charge of paving construction in a municipality is blamed for the dilapidated condition of paving, both adjoining and between the tracks of street railways, when, as a matter of fact, the entire trouble is due to the lack of co-operation

FIG. 1.—ASPHALT PAVING, TWENTY-SECOND STREET, OAKLAND, CAL. GIRDER-RAILS, STEEL TIE-PLATES, WOODEN TIES, CONCRETE TO BASE OF TIE, BASALT BLOCK LINERS ON 1½-IN. SAND CUSHION.

FIG. 2.—ASPHALT PAVING, FOURTH AVENUE, OAKLAND, CAL. VITRIFIED BRICK LINERS. ASPHALTIC CONCRETE BETWEEN TIES, INSTEAD OF HYDRAULIC CONCRETE.

on the part of the traction officials, or poor and worthless ordinances governing the control of the work, coupled with lack of support from the city fathers. Mr. Howe.

The writer makes no criticism of the construction described by Mr. Polk, but in the case of those who are about to go through the unenviable struggle of forcing public service corporations to use suitable construction to obviate ruined and dilapidated pavements adjoining railroad tracks, it may become necessary to follow an intermediate course, and in such case the construction described herein can be put down with good results. This type is not new by any means, but it embodies several unique features, such as a modern, 141-lb., 9-in. girder rail, with steel tie-plates and light concrete foundation; the substitution of asphaltic concrete for hydraulic concrete in some cases; the laying of paving blocks on their broad flat faces instead of on edge, etc., etc.

Previous to 1907, 90% of the asphalt and bituminous pavements contiguous to street railways in Oakland were in the condition shown in Fig. 1, Plate CXXXII. From time to time, efforts had been made by the traction lines to tooth the rails with basalt blocks in order to avoid this disintegration. No results were secured, as the asphalt surface broke up outside the line of the blocks. The condition was due simply to excessive vibration caused by the light T-rail used, coupled with lack of foundation and inferior roadbed. Under the existing State laws and the ordinances of the municipality, railway companies were required to pave their portion of the street, and for a distance 2 ft. outside of the outer rail, with the same class of pavement as that on the remainder of the street. Specifications governing new asphalt streets called for a concrete base of 6 in., a binder course 2 in. deep, and an asphalt wearing surface of 1½ in., making a total depth of 9½ in. for the completed pavement. Most of the rails laid throughout the city were the old type of light T-rail, ranging between 5 and 7 in. in depth. This gave approximately 2½ to 4½ in. of concrete below the top of the tie. The question as to whether the company could be compelled to increase the depth of the concrete between the ties and also line the rails with blocks, when the State laws required "that paving should be similar in all respects to the remainder of the street" was a legal one, the traction companies taking the stand that they were not required to do more than was called for by the State law. After many conferences and considerable discussion on the part of the city and traction officials, ordinances were drafted requiring the companies to line their tracks with types of paving brick or stone blocks satisfactory to the city, and to lay concrete the full depth of the ties. This type of construction was subjected to considerable criticism at the time. It was argued that the rigidity desired could not be secured unless the foundation concrete was carried to a depth of at least 6 in. under

Mr. Howe. the ties. The ordinance was adopted despite the criticism, and work was done on many miles of streets. The results have been uniformly satisfactory, notwithstanding the apparently cheap construction.

Fig. 2, Plate CXXXII, shows the west half of a completed asphalt street with basalt block liners toothing the rails. The concrete foundation on the right is complete and ready to receive the binder and surface materials. The open space adjoining the rails is ready to receive the block liners. The concrete was carried the full depth of the ties, practically encasing the wooden tie in a concrete body. Many of the ties showed signs of dry rot; the worst were removed, but many not badly affected were left in place. Owing to the depth of the rail, the basalt blocks were laid flat, instead of on edge, which considerably lessened the number used and naturally the cost. The blocks were laid on a sand cushion about 1 in. deep, and a 1:2 cement grout was

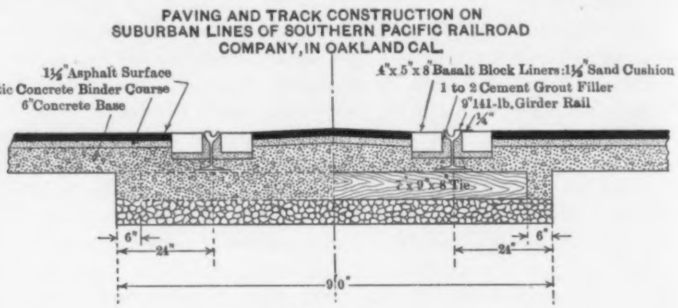


FIG. 2.

swept into the joints. During the process of concreting, the traffic was carried on one track only, cross-overs being built at intermediate points. This allowed the concrete to set thoroughly before being subjected to the strain of traffic operations. This type of construction has proven very successful, although on a street carrying the cars of four branch lines controlled by the same company.

Fig. 1, Plate CXXXIII, shows the completed asphalt pavement up to the line of the Key Route Company's tracks. On this street 141-lb. girder rails, 9 in. deep, were laid, with tie-plates as shown. The concrete extended only to the full depth of the tie. Spaces on both sides of the track are shown ready to receive the sand cushion and basalt blocks before laying the binder course and asphalt surfacing. Heavy interurban trains operate over this line.

Fig. 2 shows the type of construction on the Southern Pacific interurban lines. Girder rails of the same type were used, with basalt blocks set on edge. These blocks ranged in depth from 5 to 6 in.

Fig. 2, Plate CXXXIII, shows the type of construction used on Fourth Avenue, Oakland. Vitrified brick liners were used in place of basalt

FIG. 1.—CONDITION OF ASPHALT PAVEMENTS ADJOINING CAR TRACKS WHERE NO LINERS ARE USED. OLD TYPE T-RAIL, AND LIGHT CONCRETE FOUNDATION BETWEEN TRACKS.

FIG. 2.—ASPHALT PAVING, BROADWAY, OAKLAND, CAL. BASALT BLOCK LINERS TOOTHING THE RAILS.

blocks, and the space between the ties and to their full depth was filled with a rich asphaltic concrete instead of hydraulic concrete. The old type of T-rail used is shown. As this line was single-tracked for a considerable distance, it became necessary to maintain traffic during the paving operations. For this reason, it was thought inadvisable to put in hydraulic concrete and operate the cars at the same time. To overcome this, asphaltic concrete was substituted and thoroughly tamped between the ties, being finally rolled with a $2\frac{1}{2}$-ton tandem roller. This type of construction had been used on a number of other streets, but had not proved entirely successful. This work has been completed for several years, and when seen by the writer a few months ago was in very good condition, considering the type of rail used. Mr. Howe.

Asphalt oil bricks, similar to the Eastern type of asphalt blocks, were used as liners on several streets, but proved to be a failure. These bricks soon pounded into a uniform mass under turnout traffic from teaming, and thereby lost their individuality, taking on the general appearance of the asphalt surface adjoining. They proved to be inferior to the asphalt surface, as disintegration invariably occurred after the heavy rains, continuing during the heavy weather until the remaining portion of the block had to be removed. Asphalt blocks of better quality may obviate the earlier failures.

The construction described herein is not recommended for adoption wherever the more modern type can be secured. The best is none too good, as far as paving operations are concerned.

This type of construction has been universally adopted in Oakland, Cal., after a most successful experience covering a period of about 4 years.

Louis A. Mitchell, Assoc. M. Am. Soc. C. E. (by letter).—The writer has read this paper with much interest because similar problems are constantly coming up to be solved. Mr. Mitchell.

In the writer's experience, concrete beam construction, of which that described by Mr. Polk is a type, has not been very satisfactory for railroad tracks in paved streets. Its failure, however, has not been due necessarily to the type of construction nor to the area of the cross-section of the beam; though there is no doubt that some failures have been caused by the provision of insufficient bearing, and the fact that the loads have been too great. Most failures, however, have probably been due to loose joints which allow a slight movement of the rails when a wheel passes over them. This movement starts a hammer which the concrete will not stand, and, ultimately, the beam breaks under the joint.

By placing a steel tie directly under, and thus supporting, the joint, Mr. Polk has taken a step in advance in concrete beam construction, which, no doubt, will lengthen the life of the foundation. This sup-

Mr. Mitchell.

port will prevent the working of the ends of the rails and the hammer on the concrete. The writer believes that the life of the joint will be greatly lengthened if the support under it is increased, at least, to the full length of the joint-bar, or longer. He has used steel ties constructed of 4-in. channels, 36 in. from center to center, and connected at the ends with $\frac{3}{16}$-in. steel plates. The rails rested on these steel plates, and the joint was placed in the center so that the rails were supported for a distance of 18 in. back from the joint. Simple joint-bars 26 in. long, with six bolts, were used. These ties were fastened to the rail by castings which fitted over the base of the rail and through the steel plates (which had been previously punched) and under the rail. These castings were held in place on one side of the rail by pieces of $\frac{3}{16}$-in. plate, which were placed back of the casting and extended through the plate. The steel tie and the base of the rail were then concreted in the usual manner. This type of construction has proved very satisfactory for tracks on which 45-ton cars are operated.

The work described by Mr. Polk does not deal with the most important part in the making of a good joint for paved streets, although he may have taken care of this properly. When bolts are used, the holes in the web of the rail and in the joint-bars should be of the same size as the bolt, and machine-bolts should be used, making a driving fit. The ends of the rails should be ground so that they will fit tightly, especially at the ball. The drilling should be such that when the bolts are in place the ends of the rails will fit tightly together. This type of joint allows for no change of length in the rail due to change in temperature; but this is not necessary owing to the fact that in paved streets only a small portion of the rail is exposed to extreme changes in temperature, and the tendency to change in length is taken up in internal stresses.

Any open joint will permit the wheels of a car to pound, and this will increase until the ball of the rail is ruined. This pounding may be maintained at a minimum, even in open joints, by keeping the rails ground so that they are of the same height at the joint.

Another cause of pounded joints, and one which will produce failure just as surely as an open or loose joint, is difference in the height of the rails. Specifications for rails allow for a possible variation in height of $\frac{3}{64}$ in. at any joint, and even if the joint is tight in the bolts and the rails well fitted, a pound will start, and there will be cupped rails in the track in a surprisingly short time. For this reason, the joints of new track should be ground to an even surface just as soon as possible after the work is completed.

In track constructed as described, the writer has seen joints which were ground soon after the work was completed, and it required a rather close inspection of the ball of the rail even to find where they were.

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The writer's work, however, is not of concrete beam construction; in some cases there is concrete under the whole track, and in others the track is on broken-stone ballast with concrete from the bottom up to 2 in. above the top of the ties. The track constructed by both these methods and with joints of the type previously described, has required no maintenance whatever since it was installed $1\frac{1}{2}$ years ago, but it was found that, in some cases, even with the tight joints, the rails had cupped, due to the difference in height, and it was necessary to grind the joints to an even surface.

Another method of constructing a tight joint is similar to that described, except that rivets are used instead of bolts, thus assuring a tight fit between the rail and splice-bar.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

THE FLOOD OF MARCH 22D, 1912, AT PITTSBURGH, PA.

Discussion.*

By Messrs. L. J. Le Conte, and William R. Copeland.

L. J. Le Conte, M. Am. Soc. C. E. (by letter.)—The writer is deeply interested in this paper, because the general scheme for flood control therein considered was, in his younger days, a pleasing day-dream which haunted him for years. Long experience in this line of thought, however, has brought a realization of many practical difficulties which cannot be easily adjusted. The author says: Mr. Le Conte.

"The system of storage reservoirs could be operated primarily for flood prevention during the flood season, and for increasing the low-water flow during the low-water season."

This is perfectly feasible, and true in every respect, provided every private or quasi-private interest is prevented from interfering with free and untrammelled operation in the interest of flood control purely.

The author also states:

"The benefits to navigation, sanitation, water supply, and water power, which would result from such an improvement in stream regimen would naturally be very considerable."

This is exactly where the fundamental difficulties of the whole scheme come in. Experience everywhere shows that it is almost impossible to reconcile private interests and flood-control interests in one and the same scheme. The conflict is irrepressible, and, in a majority of cases, the combined scheme is utterly impracticable. A single instance is sufficient to show the true nature of the irrepressible conflict. The same reasoning will apply to any one of the seventeen reservoirs proposed.

* This discussion (of the paper by Kenneth C. Grant, Assoc. M. Am. Soc. C. E., published in August, 1912, *Proceedings* and presented at the meeting of November 6th, 1912), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Le Conte. Complete flood control requires that all the storage reservoirs shall be practically empty just before the expected peak flood arrives, say, in February and March of each year. This will furnish the desired storage room for the great rush of flood-waters, and everything will work satisfactorily. On the contrary, how does this requirement affect private interests? Water-supply and water-power interests both demand that the reservoir shall be filled as early in the wet season as possible so as to be absolutely sure of a full reservoir before that season is over. This means that the expected great "peak flood," which comes in February and March, will rush down the river and into a full reservoir, pass over the crest of the dam unrestrained, and continue down stream just exactly as it did before the dam was built. As a result, the flood height in the lower river would be the same as before the reservoir system was built, if not greater, inasmuch as the proposed improvements are supposed to raise the low-water plane of the river to some extent.

From this it is clear that, as a rule, flood-control schemes cannot be combined with such schemes as water supply, water power, irrigation, etc., on account of conflicting interests which are almost insurmountable. This is the principal reason why this truly fascinating problem has remained dormant for sixty years or more, and has never received a practical solution.

The writer ventures to submit a few mild suggestions, which, of course, pass for what they are worth. Where the flood-danger period does not extend over 2 or 3 months each year, the storage reservoir waters could be used for power or water-supply purposes during the remaining 9 or 10 months. During the 2 or 3 months when the storage reservoirs must be kept practically empty—in anticipation of the great peak flood—all the power plants and water-supply plants must necessarily be kept going with auxiliary steam plants erected for the special purpose. The long transmission lines in California generally have auxiliary steam plants in the cities where they sell their power and light. These auxiliary plants are started up whenever there is a breakdown on the main transmission lines; therefore, a stoppage of 2 or 3 months at the main power-house is not vital. Likewise, in the case of a water-supply company: if the natural flow of the stream above the reservoir site, in February and March, is sufficient for water-supply purposes, all well and good, and no steps need be taken for an auxiliary supply; but, if it be short of requirements, a small auxiliary steam pumping plant could be located just below the dam site, the pump wells being fed by underground seepage from the reservoir above. The capacity of this plant need be only sufficient to cover the deficiency.

Mr. Copeland. WILLIAM R. COPELAND, ASSOC. AM. SOC. C. E. (by letter).—Mr. Grant evidently prepared his paper with the idea of proving that the construction of storage reservoirs on the water-shed of the Allegheny

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River will protect the residents of Pittsburgh and vicinity from damage by flood. He seems to have overlooked an important matter in this connection, and that is the question of the effect which may be produced on the Allegheny by the storage of acid waters from coal mines in such reservoirs.

Consider, for example, the proposed reservoir on the Loyalhanna. This river, rising on the western slope of the Allegheny Mountains, flows northwestward for about 30 miles until it joins the Conemaugh at Saltsburgh, forming the Kiskiminetas. From its source to Latrobe, it lies in a rather broad, open valley. Between Latrobe and Saltsburgh, however, it winds through a valley, so narrow and deep in many places that it becomes a gorge.

This valley is crossed at four or five places by dikes of stone which form natural dams, and they have been raised in height artificially for the purpose of storing water for mill powers. The pools thus formed—each a mile or more in length—serve as catch-basins for the mine drainage entering the stream from each side.

This mine water has several marked characteristics, one of which is that its specific gravity is greater than that of ordinary surface water. Therefore, the drainage drops to the bottom of the river, collecting, of course, in the pools above the dikes. Another bad feature of mine drainage is the free acid and acid salts which it contains.

Chemical analyses have shown that water flowing from coal mines in this region carries from 100 to 500 parts per million of free acid. Dilution and reaction with the alkaline carbonates carried by the surface waters tend to reduce the acidity, but, nevertheless, a sample of water taken from the bottom of a pool in the Loyalhanna in September, 1899, contained 200 parts per million of free acid—and the sample was taken just after a considerable flood had passed down the stream.

July, August, and the first half of September had been very dry, but about September 15th a thunder-storm having the characteristics of a cloudburst broke upon the upper water-shed. So great was the rainfall that the run-off raised the river more than 2 ft., creating a current in the Loyalhanna which swept the immense volumes of acid mine drainage stored in the pools into the Kiskiminetas, and eventually into the Allegheny.

Millers and farmers have complained for years that the mine waters of the Loyalhanna eat up their iron water-wheels, and poison stock which drink from the stream. One man told the writer that he had lost a 1-in. iron crow-bar through the ice on the river during the preceding winter, and that by spring the bar had been eaten in two.

The head-waters of the Loyalhanna contain fish, frogs, lilies, and

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all the natural aquatic life of the region; but below the entrance of the first coal-mine drains, at the outskirts of Latrobe, not a fish nor a weed can be found in the water.

When the mine water—swept into the Kiskiminetas by that September flood—reached its month, the acid burned the legs of Italian laborers working on a bridge pier, and drove them out of the river. Passing into the Allegheny the sulphur water killed tons of fish in its run of 40 miles to Pittsburgh, and the rotting bodies created such foul conditions that the Superintendent of the Pittsburgh water supply at once started an investigation regarding the cause of the death of the fish.

Smaller floods of this character are of common occurrence in the Allegheny, and are becoming more common annually. The volume of mine water is increasing, too, as new coal mines are being opened yearly, the workings in others are being extended, and last, but by no means least, the drainage from all former openings continues to pour unceasingly into the watercourses.

At present the dikes and dams are so low that the Loyalhanna is flushed out several times a year, but what will happen if a great reservoir is formed by throwing a dam across the stream near its mouth, as indicated on the plan, Fig. 2. Such a structure will surely store the acid mine drainage, and, if any exceptional flood, from a cloudburst or other severe storm, sweeps the water out of the reservoir, far greater volumes of stronger acid water will be poured into the Allegheny than has ever entered it at one time before.

When this happens, the water consumers in all the towns on the river, and the men in charge of the water purification works at Pittsburgh and elsewhere, may well take heed lest the mine drainage destroy their boilers and water mains, and even close the water-works plants temporarily.

The location of the dam on the Loyalhanna has doubtless been chosen in order to get a maximum amount of storage and to decrease the danger of flood from this region to a minimum; but, in view of the presence of mine water on the lower water-shed, is it advisable to locate the dam at the proposed site? Would it not be better to build the dam above Latrobe, for instance, at Ligonier? Part of the town, a railroad, and some highways would doubtless be wiped out, and the volume of water stored would be reduced. Some persons might argue, further, that the process of holding back the alkaline surface water at Ligonier would cause a more concentrated sulphur water to flow into the Allegheny. This effect could be easily remedied by opening gates at the dam, and really improve present conditions by keeping a larger and more uniform flow through the lower valley at all seasons.

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If the dam is ever built at the site proposed in Fig. 2, channels should be cut through all the present dikes to drain the heavy acid water from the lower parts of the pools; and blow-off gates should be placed in the bottom of the dam through which a constant discharge of water will be maintained for the purpose of preventing the acid mine drainage from destroying the gates or even injuring the structure itself, as well as the city mains and plants on the lower rivers.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

STATE AND NATIONAL WATER LAWS, WITH DETAILED STATEMENT OF THE OREGON SYSTEM OF WATER TITLES.

Discussion.*

By Messrs. Clarence T. Johnston, L. J. Le Conte, George L. Dillman, W. E. Moore, Morris Bien, Horace W. Sheley, and Morris Knowles.

Clarence T. Johnston, M. Am. Soc. C. E. (by letter).—This paper deserves some comment from those who are interested in problems of stream control. The writer is glad to find, here and there, an engineer who has had sufficient experience in such work to convince him that the questions which arise are of a legal nature only in an incidental way. In its fundamental aspects, the supervision of water resources is of greater public concern than the protection of land titles. Water and watercourses are generally considered as public property, and, therefore, the demand for an engineering administration has not been felt, except where streams have been used to a large extent, and particularly where water has been diverted from natural channels. As controversies have arisen, the Courts have been appealed to, and to-day, particularly in the West, decisions of all kinds are made. Volume after volume is published dealing with theories which have been developed and played with by attorneys, until they can no more be applied in practice than water can be diverted from a stream without affecting its discharge. Mr. Johnston.

We derive our common law from England. With that law we brought the doctrine of riparian rights, which guarantees to every owner of land abutting on a watercourse the right to demand that

* This discussion (of the paper by John H. Lewis, Assoc. M. Am. Soc. C. E., published in September, 1912, *Proceedings* and presented at the meeting of November 6th, 1912), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Johnston. the waters thereof pass his property "undefiled in quality and undiminished in quantity." This is the true riparian doctrine. It is not suited to a country where large volumes of water must be taken from the streams. Regardless of this and regardless of the experience of other countries, lawyers and Courts have tried to make it fit every climate and every natural condition. A simple doctrine, which suits a country where large rivers prevail and where actual diversions are of little importance, has been distorted and juggled with by the Courts until it is no longer recognizable. The "modified" doctrine of riparian rights, as we have it inflicted on us, is often a convenient screen for crimes committed against the public and in behalf of those claiming "vested rights." A perusal of the statutes of various States referring to riparian rights, and quite generally the laws pertaining to stream control, will disclose the existing situation to the student. Much has been done to protect vested rights, but reference is seldom made to public rights. The idea that the public ever obtains a vested right would never be gleaned from reading the statutes dealing with stream control in a majority of the States. This condition arises from the fact that the lawyer, representing those who claim many special rights and privileges, often reaches the Legislature where he exhibits marked ability in inserting such clauses as the following: "This chapter shall in no wise be construed as impairing or abridging any rights already vested in any person or persons, company, or corporation, by virtue of the law heretofore in force." After a training of this kind he develops an instinct which enables him to protect private property rights without study or deliberation. Should such a man be elevated to the bench, he displays the same tendencies, and some of our remarkable decisions may be attributed to Courts so constituted.

Streams and lakes are not like land, in so far as private possession is concerned. Land can be measured and privately controlled. Water is constantly shifting, and the supply changes every day. The public must bridge streams and provide harbors. The public must protect fish and provide for the safety of dams. We use water to-day only to lose it to-morrow when it runs on, a continuous blessing to the public. It cannot be owned privately, and no State should ever permit individuals or corporations to claim such ownership. It is plain that injustice would be worked should one nation have exclusive ownership of one of the great oceans. It is equally plain that injustice would result should one State control an interstate stream or lake. The same rule applies when one individual or corporation is permitted to assume control of any local water supply, yet the local public is often slow to act in its own defense. The engineer should appreciate these facts. He should be a leader in questions of stream control. Unfortunately, engineers have not assumed the responsibility that naturally belongs to their Profession. We may criticize the Legal Profession

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and the Courts—and possibly they deserve it in some degree—yet we must remember that the engineer is depended on for information; and further, that where engineers have studied these important problems and given the Courts the results, reforms favoring public control have begun to appear. Too many engineers have been blinded by what seems to them to be the best policy at the time for their employers who appeal to the Courts. Too many engineers fail to see that principles which insure justice to the public protect water users generally. The water rights of private users are best insured where public control is most rigid. Engineers have been as tardy as the Courts in recognizing this.

When the National Constitution was framed, the States ceded to the General Government the control of navigable waters. The nation, as a matter of development and defense, must control navigation when necessary. Non-navigable streams remain in the possession of the States. The States have not properly administered the smaller streams which they own. It has been so apparent that streams and lakes are naturally property of a public character that, for a long time, particularly in localities of large rainfall, the necessity for public supervision has not been called to the attention of lawmakers. Regardless of the public character of streams and lakes, private interests have assumed to take possession here and there, and the claims thereby arising are still to be settled. The great questions relating to State administration of waters are yet to be determined. Some Court decisions would appear to be final and in favor of a private monopoly of public property of this kind, yet the end has not been attained in a single important case.

Because the States have been slow in asserting the doctrine of public rights in property which is essentially of a public character, we need not fear that the public has lost title in any way. Because some individual or some corporation has been using this property for a term of years, there is no reason to hold that the users have secured title thereto. It would seem that such users would owe the public a vote of thanks at least. Many troubles have been precipitated because the owners of riparian lands have been permitted to claim more than the true riparian doctrine would admit in itself. Because the General Government has not meandered every small stream—a physical impossibility—the subsequent patentee of riparian lands has assumed that he owns the land to the center or entirely across the channel of the natural waterway. Where, through ignorance of the naturally public character of streams, the Courts have seemed to confirm claims of this nature, an error has been made, but such errors will be corrected in time.

Water must be used by individuals and by corporations. The public should realize this and consent thereto. The public and all users

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should appreciate the difference between the use of water and the ownership of water. All uses should be of such character, and secured under such restrictions, that the authority and ownership of the public are manifest in every transaction and apparent to all concerned. When the time comes for the public to assume full control of its property, private interests should be recompensed according to the estimated cost of replacing the works constructed by them, and which, under the new order of things, are to be operated by the public. Valuation work of this kind has also fallen largely to the engineer. It is a new science, and engineers have not reached an agreement as to the principles which should be applied in determining just valuations.

Principles are of so much more importance than details of law, methods of procedure, or exact character of administration, that the writer does not feel inclined to discuss National, State, or even more localized control of streams. Under present conditions, there can be no question as to the responsibility of the General Government and of the States in work of this kind. The essential facts which should determine any question which may arise are of an engineering, rather than of a legal, nature. They are simple and, as a rule, not difficult to obtain where an engineering administration is provided.

The writer has followed the development of the public control of streams of the West for many years. Wyoming, under the able leadership of Elwood Mead, M. Am. Soc. C. E., took the first step which relieved the Courts from all initial proceedings in questions relating to the use of water. Only appeals from administrative officers go to the Courts in that State, and these appeals are so few that during the past twenty years they can be counted on the fingers of the two hands. During that time the State Administration has studied and determined more than 15 000 claims and, at the same time, has protected all public rights. As Dr. Mead's Assistant for a term of years, and as State Engineer for nine years, the writer was able to give some thought to the principles which should underlie the laws relating to stream control, and the following matter from his last report* to Governor B. B. Brooks contains a discussion of the elementary principles:

"Some reference should be made at this time to Sections 724, 725, and 726, Wyoming Compiled Statutes, 1910. These sections were added to the irrigation laws of the State by the Legislature of 1909. The bill before the Legislature was known as 'House Bill No. 66.' All who have studied this legislation agree that it represents the most important action of the law-makers of the State since the original statutes were enacted in 1891. While the purpose of the act is fresh in the minds of those who were in the Legislature and among those who prepared the bill, some public record should be made of its purpose so that when its provisions are interpreted, there will be no mistaking its object.

* Biennial Report of the State Engineer to the Governor of Wyoming, 1909-10, pp. 17 to 29.

"The Act reads as follows: Mr. Johnston.

"CHAPTER 58.

"*Water Rights.*

"724. *Water Right Defined.* A water right is a right to use the water of the state, when such use has been acquired by the beneficial application of water under the laws of the state relating thereto, and in conformity with the rules and regulations dependent thereon. Beneficial use shall be the basis, the measure and limit of the right to use water at all times, not exceeding in any case, the statutory limit of volume. Water being always the property of the state, rights to its use shall attach to the land for irrigation, or to such other purpose or object for which acquired in accordance with the beneficial use made and for which the right receives public recognition, under the law and the administration provided thereby. Water rights cannot be detached from the lands, place or purpose for which they are acquired, without loss of priority. (L. 1909, ch. 68, Sec. 1.)

"725. *Preferred Uses Defined.* Water rights are hereby defined as follows according to use: Preferred uses shall include rights for domestic and transportation purposes; existing rights not preferred, may be condemned to supply water for such preferred uses in accordance with the provisions of the law relating to condemnation of property for public and semi-public purposes. Such domestic and transportation purposes shall include the following: First—Water for drinking purposes for both man and beast. Second—Water for municipal purposes. Third—Water for the use of steam engines and for general railway use. Fourth—Water for culinary, laundry, bathing, refrigerating (including the manufacture of ice), and for steam and hot water heating plants. The use of water for irrigation shall be superior and preferred to any use where turbine or impulse water wheels are installed for power purposes. (L. 1909, ch. 68, Sec. 2.)

"726. *Change to Preferred Use.* Where it can be shown to the board of control under the provisions hereof, that a preferred use is to be made, the procedure for a change of such use shall embrace a public notice, an inspection and hearing if necessary by and before the proper division superintendent, a report of such superintendent to the board of control, and an order by said board. If the change of use is approved, just compensation shall be paid and under the direction of the board, proper instruments shall be drawn and recorded. (L. 1909, ch. 68, Sec. 3.)

"The act is plain in its terms. There may be some confusion, however, in the minds of those who have not followed the history of the development of irrigation law in this State. The constitution of the State which has the approval of Congress, says:

"'Section 1, Article VIII. The water of all natural streams, springs, lakes or other collections of still water, within the boundaries of the State, are hereby declared to be the property of the State.'

"This provision of the constitution has been discussed from various standpoints. The State has never held that the water which it owns should be disposed of for profit. It is not deemed wise to administer this natural resource for revenue. It is presumed by the law-makers of Wyoming that the water the State possesses is for the use of the

Mr. Johnston. people. However, the State has a greater responsibility. Wyoming has discovered and placed on the statutes of the State the first definition of essential principles that should govern the use of water. So that the State, not only owns the water and permits of its use without charge to her citizens, but the State sees to it that all users are given equal protection. One man cannot be permitted to expand his use without interfering with the rights of a community. The State early discovered the danger of permitting water or water rights to be treated as personal property. If water rights are owned separate from the land irrigated, or such other use as may be made of the water, then the personal element enters and there is no such thing as public supervision and no such thing as the protection of the community against the greed of a few who may be powerful or influential.

"We cannot review the decisions of the Colorado courts without feeling that they have failed to enunciate such doctrines as will protect the water user. For instance, we find in the case, 'Laramie County Res. Co. *v.* People *ex rel.* Luthe,' 8th Colo. 614, the following definition of 'appropriation.' 'Appropriation is the intent to take, accompanied by some open, physical demonstration of the intent, and for some valuable use.' The court evidently understood that a diversion from the stream without actual beneficial use of water could not give the right to use water. It was manifestly plain to the court that there was something necessary in addition to a claim, which we assume to be the manifestation of the 'intent.' The court did not see that an investment in irrigation works can be protected without giving the investors ownership in water. It saw that a time must lapse between the filing of the claim, or the expressing of the 'intent,' and the use of the water. Hence the priority would date from the time the claim was filed. The company, unless disturbed by some agency, would secure title to water under that priority date providing it completed the irrigation works. There was no limit to the demands the company might make on the stream, except its financial ability in ditch and reservoir construction. There was nothing in the plans submitted to tell where the prospective beneficial use was to occur and absolutely nothing to protect those who were to be the actual water users. It might have occurred to the court that the State could by law provide that a company be given a certain time to show its good faith and upon the final showing being made, that it be given a certain time to sell interests in the irrigation works constructed. It might have occurred to the court that this would have enabled each water user to have obtained water rights dating back to the time the claim was filed in some office of public record; it would have enabled the company to have obtained a reasonable profit; it would have left the water rights in possession of the water users, each having an interest in proportion to the land reclaimed; it would have given the irrigation works to those who should for all time rightly be held responsible for maintenance. How the construction of irrigation works without any arrangements being made with prospective users or without any reference to the proposed beneficial use, can give the builders a right to the water is a problem that is too complicated for the average man to grasp. The court saw the danger ahead and it tried to do something to avoid it. The trouble was that the court gave the company every-

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thing in sight, under certain conditions. It is like giving a railroad an entire valley on condition that transportation facilities be provided. The railroad has a right to demand lands which it uses or is to use. The ditch company has this right. Water is more valuable than land. The railroad company is not entitled to all the land in sight, when it cannot use the land. The ditch company cannot use all the water it can divert. It has no right, therefore, to be placed in position where it can sell water which it has never acquired by beneficial use. A railroad company which secures an entire valley receives a rich reward. It can sell the lands to those who can cultivate them. This arrangement should be very pleasing to the railroad company. It was doubtless as pleasing to the ditch company to get title to water. It could sell this water to those who either had to buy it or face financial ruin. There is absolutely no reason why an irrigation company, or any individual, should have title to any property except the physical works it constructs or acquires. The court did not see this. It did not see how the water users could be protected under the priority obtained by the initial filing except by giving the company title to the water. This is so simple and it works out so easily where the States have an administrative system which manages streams for the benefit of the public, that any other plan would never be discussed if it were not that living examples of a dangerous type are so close at hand. The Colorado court decisions go a little further than to give everything to the company which builds the irrigation works. The rights of the consumer are discussed in a number of cases. This admits of some intimation of rights that others, aside from the irrigation company, might hold. If the court, in the first place, had held that the plans submitted by the company must be based on good engineering and that specifications for every structure must be filed; if it had then compelled the company to designate the lands that were to be reclaimed and dedicated the water to the benefit of those lands, a foundation would have been laid that would have supported every principle that might be necessary to protect those that were to follow the company. There is a library of speculation over this subject of appropriation in the Colorado decisions—yet no two complete discussions are in entire harmony. All indicate that there is something that should be reached and in every decision this something is just beyond the grasp of the court. To prevent too much injury being done the water users, the State Legislature has given the county commissioners authority to regulate the price paid for water. Because the court could not fathom the problem before it, the future of every project is left with administrative officers, who may or may not be able to determine what price is fair. The county commissioners may be connected with the company and they may be water users under the system in question. It would seem much better to make the water rights attach to the land and establish the price which the water user is to pay for an interest in the irrigation works in the beginning.

"Let us study an individual case so that its history may be approximately complete. The Colorado courts dealt out water, rather than water rights, with a lavish hand. The courts did not know what volume of water was necessary to satisfy with justice the needs of each claimant, and they did not know what principles should govern

Mr. Johnston. in the adjudication of rights. No person can arrive at any other conclusion after reading closely the court decisions of the State. For instance, a water claimant is given a right to 35 cubic feet of water per second. The court does not say that this water belongs to the land or that the water right belongs to the land, but the person is given a kind of title to so much water. As the country grows the person so favored finds that the court has given him a large volume of water that he has never used. He therefore applies for a change in point of diversion. This may be opposed in the courts, but he generally wins out. He then proceeds to use this water on new lands, water that others have for years past been using. Those who are injured may be able to protect themselves if they have means. If not they must suffer the consequence. This person, who is so fortunate to get possession of water that he has never used, then proceeds to use it in another way that injures the community still further. Because the court has given him 35 cubic feet of water, he holds that he can store all of the excess that he does not need in the irrigation season for direct application from the stream. This is a happy thought. In the meantime possibly enough reservoirs have been built to store all of the excess flow of the stream. This makes no difference to this fortunate person. He stores, for his own use and for speculation, 35 cubic feet of water per second, except during the irrigation season, thus depriving other persons of his community of water that they have been using.

"He has probably secured the 35 cubic feet of water per second when he only irrigated 160 acres of land. Two to two and one-half cubic feet per second during the irrigation season would be ample for this area. Even should he be able to apply three cubic feet per second, he would have left 32 cubic feet per second for speculation during the irrigation season. This would supply water for 2 000 acres of land and be worth $80 000.00 at least. By storing 35 cubic feet per second for nine months he would impound 18 900 acre-feet of water. If a fourth of this, or 4 725 acre-feet of water are lost by seepage and evaporation, he would have left 14 175 acre-feet of water for further speculation. This would irrigate fully 6 000 acres of land and be worth $240 000.00. The court, therefore, through ignorance, and without considering the rights of any person except the claimant, gives him $320 000.00 worth of water that he has never used and which has been used by others. This value of $320 000.00 cannot be given to one claimant without taking it from another.

"What is the situation in Wyoming? An agricultural community begins to develop. Each applicant for water rights obtains permits which describe lands to be irrigated. The water rights are finally adjudicated and dedicated to the use made. The water used for irrigation belongs to the lands reclaimed. It remains attached to that land. When the land is sold it goes with the land. The law places a maximum limit on the use that can be made—one cubic foot of water for each seventy acres of land irrigated. The public must see to it that waste does not occur. This maximum limit cannot be exceeded at any time and the use is restricted to such volume less than the legal limit as can be beneficially applied to the lands to which the water right attaches. There is no way to expand the use. There is no way by

which an individual or a company can speculate at the expense of a community. All rights are matters of record and there can be no serious cause for dispute. Mr. Johnston.

"Because certain lands have water rights direct from the stream during the irrigation season does not mean that these lands are entitled to water throughout the year. The first reservoir built in compliance with law secures the first right to store water to its capacity. This reservoir can be filled at any time that the stream furnishes a supply that is not used. The second reservoir then has its turn, and so on. Reservoirs store water—not water rights. Ditches carry water—not water rights. Water stored in reservoirs simply augments the supply during the season when needed and this water leads to the perfection of water rights which belong to the lands reclaimed and to the other uses made, the same as though the stream supplied an ample volume during such seasons.

"Section 724 states that 'beneficial use shall be the basis, the measure and the limit of the right to use water AT ALL TIMES not exceeding, in any case, the statutory limit of volume.' This means that when the community needs protection the public control must be such as to limit all users to the volume that can be beneficially applied. The certificates of appropriation, issued by the Board of Control, describe the lands to which the water right belongs and specifies what the maximum allotment of water is to be. It also says in conformity with the law that the use shall consist of the application of such part of the maximum allotment as can be beneficially applied.

"Water is the property of the State and no charge is made for its use. When water is used for irrigation or for any beneficial purpose for a term of years, the user should feel thankful to the public that conditions are such that he has been able to do business without being subjected to a special tax on the water. Because the public has enabled this to be done, is there any reason why any person or company should be given the additional right to sell the water or water right he has been enjoying? The community cannot be injured as long as the use continues as it has always been, but that use cannot change without having some effect on the rights of others. Because the last sentence of Section 724 states that water rights cannot be detached from the lands, place or purpose for which acquired without loss of priority, it is held by some that the act is unconstitutional. Under the constitution cities and towns can condemn water rights for municipal purposes. When a city or town needs water it gets it and always gets it regardless of priority or the character of the right. Under this same act, municipal uses are made preferred uses. It may often serve the purpose of a town to condemn a water right that belongs to some land not enjoying an early priority. For instance, the early rights may all be near the source of a stream. This is an ideal condition. The town may be twenty miles below the lands having these rights. There may be irrigated lands near the town that can be condemned to furnish a water supply for the municipality. The water right obtained by the city or town under such circumstances cannot affect the early rights at the head of the stream, yet the town has secured an adequate supply. Should it be necessary to condemn one of the early rights at the head of the stream, this could be done, at any time later.

Mr. Johnston. The idea is this—nothing should be done by the State to give an added value to the early rights. There are so many conditions affecting the flow of a long stream, that cities and towns can often be supplied without interfering with the early irrigation development. A town might buy an adjoining farm with whatever water rights it may have. If the use of this water for municipal purposes will injure other water users, outside of the town, then they should be recompensed. After a city or town buys water rights, they become preferred uses, so that the priority of the right makes but little difference. The main object is to see to it, at the time the transfer is made to the town or city, that all who might be injured by this change are taken care of at once. Section 726 provides that these changes shall take place under the direction of the Board of Control. This means that an inspection will be made by the Superintendent, all users will be notified and the final record will embrace a settlement that will be in harmony with physical conditions and with justice to every allied interest in the stream. One thing should be remembered—preferred uses are to be protected in seasons of scant water supply. When a change is made of any inferior right to a preferred use, all who might be affected should be made acquainted with the purpose of those who represent the preferred use and all precautions taken while the matter is before the board to secure whatever redress may be justifiable. It should also be remembered that preferred uses require small quantities of water compared with uses for irrigation. For instance, suppose a man owns 160 acres of land to which a water right attaches. Assume that he covers this to a depth of two feet during an irrigation season of 90 days. This is 320 acre-feet of water, or 104 544 000 gallons. This volume of water would furnish a water supply for a town of 3 000 people for a year, estimating that each person requires 100 gallons of water per day of twenty-four hours. Since the irrigator does not use water from the stream, except during the irrigation season of say 90 days, he has no right to the water during the remainder of the year. The town can file an application for the necessary supply outside of the irrigation season and the permit secured gives rights that cannot interfere with the rights of any other user, unless other rights have been secured which depend upon diversions outside of the summer season. The water right for 160 acres with the additional flow that could be secured under permit would supply a town of 12 000 people, if the water could all be used without loss. Certainly this should be a preferred use. The value of a city of 12 000 people to the State and to the public as locally represented, is far in excess of the value of any 160-acre farm, no matter how highly it may be cultivated.

"Another point should not be overlooked in this connection. The State does not permit the use of the legal maximum flow—one cubic foot per second for each seventy acres of land reclaimed, except when this volume can be beneficially applied. The owner of irrigated land cannot charge except for the value of the land with a water right, compared with the land in its former arid condition. He cannot, in other words, transfer to a municipality or to any preferred user, the maximum limit of his water right. This must be fixed by the board of control at such hearing as it may conduct. The city may buy the land outright and the entire tract of say 160 acres may be irrigated. The

land may be of such character that one acre-foot per annum will raise crops and this volume may have been the maximum that has ever been used. Manifestly this is all the municipality obtains.

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"It is believed by those who understand natural streams and the effect of diversions therefrom that it is much better to permit preferred users to purchase any rights that may be available rather than to place a premium on the first right, as has been the custom. For instance, if the first right were at the head of the stream and the city at or near its mouth, the irrigators above would lose all return seepage from the lands irrigated under the early right should this be transferred to the municipality. The early right and all other rights might continue undisturbed should it be possible for the town to secure a comparatively inferior right nearer to its boundaries.

"This legislation was not prepared on the spur of the moment. It represents the result of many years of study. It received the consideration of irrigation authorities throughout the West. It was given very careful study by the committees on lands and irrigation and by individual members of the Legislature of 1909. This act does not represent the personal views, only, of any individual or by any class of individuals. It represents in concrete form the wisdom of water users, students of irrigation and legislators. It passed the Legislature of 1909 with but seven dissenting votes altogether. No attempt was made to frame a measure of the kind until the views of the water users of Wyoming had been obtained. The responses received to letters mailed to thousands of irrigators throughout the State laid a foundation for the bill as it was submitted.

"It has been said that other States and that courts do not recognize the principles embodied in this act. It must be admitted that Wyoming stood for something of the kind long before any such doctrines were heard of outside of the State. It must be admitted that because other courts failed to get down to the proper fundamental principles, our courts had no guide, except in so far as our law-makers prepared the way. However, these doctrines are spreading. We need not search in vain for decisions that have the fundamental principles clearly defined. On March first, 1910, a decision was handed down in Arizona which should be read in full by every student of irrigation. The title of the case is 'Patrick T. Hurley, the United States, Intervenor, *vs.* 4 800 other water users.' It relates to the settlement of claims to use water from Salt River. The Reclamation Service is constructing one of its large projects in the valley of this stream, hence the United States intervened in the suit. The Supreme Judge of the Territory, acting as District Judge, heard the evidence and made the decision. It is remarkably complete. The definition of principles appearing on pages 8 and 9 of the printed copy of the decision is worthy of careful study and consideration. We quote this in full as follows:

"'The doctrine of riparian rights does not obtain in Arizona. The right of the owner of land to divert from a natural non-navigable stream the flow of water therein and to apply the same to beneficial use upon such land, is and always has been recognized in this territory. Such diversion and use is termed an appropriation of water. Whatever may be the steps necessary to take to initiate such a right or to evidence the intent to initiate it, the appropriation itself only becomes complete

Mr. Johnston. and vested when the water is actually diverted from the stream and placed to a beneficial use upon the land. The right given by such an appropriation is strictly not a right to the water itself, but a right to the use of the water. Its application to a beneficial use upon the land is as necessary in order to complete the right as is the diversion thereof from the stream. An appropriation of water, therefore, for the purpose of the irrigation of a parcel of land may not be established and completed by means merely of a declaration of intention or by the posting of notices of appropriation, nor may it be made by a canal owner or by a canal company as such alone, independent of its ownership of the land; but as application to a beneficial use upon the land is necessary to complete the appropriation, it follows that such appropriator must be an owner of land or have a possessory right thereto. Furthermore, since the land to which the water is to be applied is a necessary integral part of the appropriation and a factor by which the amount of the water appropriated for use is measured, it follows that when the water is no longer applied to the land for which it was diverted, the right of appropriation of such water for such land ceases. The right of appropriation further depends upon a supply of water that is unappropriated. It follows, therefore, that the first in time of appropriation is the first in right to appropriate, since water previously appropriated by another is no longer available for a subsequent appropriator. The extent of the appropriation is limited by the beneficial use to which the water can be applied. The actual amount of water that may be appropriated for irrigation, therefore, is the amount that the land owner can and does actually use in the necessary and economical irrigation of his land for cultivation. This much and no more may he have; and this much he may only have when there is sufficient water available to supply first those prior in date of appropriation. The fundamental principle in the doctrine of appropriation of the normal flow of water in a stream for irrigation is its application by the land owner to the land for a beneficial use. The right to appropriate is a right that belongs to the land owner, but the water appropriated is appropriated for the land, and when so appropriated its use belongs to the land and not to the appropriator. The method of diversion from the river and the means of carriage of the water to the land is immaterial in the establishment or maintenance of the right; it may be done by the individual appropriator or by an association of individual appropriators, or by a canal company, or by any person or corporation; and the means of carriage or the point of diversion from the river may be changed from time to time to suit altered conditions without impairing the right of appropriation already made, provided prior rights of others are not interfered with. There being in this territory no private property in water, but water being a public property subject to the uses before defined, in so diverting and carrying the water such person, association or corporation acts merely as the agent of the appropriator and acquires no right of appropriation to the water itself, and no rights as against the appropriation made to the land, except a right to proper compensation for such diversion and carriage.'

"This decision marks distinct progress in court decisions relating to the use and diversion of water from natural streams. The principles

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upon which the decree is rendered afford protection to every claimant yet when they are applied in practice no man is given a weapon whereby he may destroy the prosperity of his neighbors. The personal element is eliminated. The welfare of the community, through the protection of each individual in accordance with the nature and extent of beneficial use of water, is taken as the criterion for the settlement of rights rather than the claims of individuals regardless of the character of the irrigation development they may have been responsible for. Because the federal government intervened in this case and because the judges were at the time federal officers, the decision has a value that is more far-reaching than would ordinarily be the case. The entire decision is recommended to the consideration of those who are interested in the settlement of water right claims and controversies.

"It has been held by those who do not support the principle which unites water rights to the land reclaimed that the act of 1909 does not apply to water rights which were perfected prior to the passage of the law. This is the old argument that supports 'vested rights' regardless of the welfare of the public. The public and communities generally will protect individuals in their rights if the public and the community is made supreme over the individual. If the individual is superior he will take advantage of his position to injure the community. If the State cannot make laws affecting the administration of its own property, then it certainly cannot enact statutes which have for their purpose the collection of taxes, for instance, which relates to the raising of revenue from property in private possession. It cannot quarantine live stock belonging to private parties even for the protection of communities. It cannot pay bounties for killing predatory wild animals or defray the cost of protecting game animals. To say that the State cannot regulate the use of water by the enactment of new legislation, even after water rights have been acquired for various purposes, is on par with the claims of certain people who held that because they began the use of water prior to the admission of Wyoming to statehood, the State had no control. This contention led certain claimants in Johnson County to refuse to submit proof before the Board of Control and to attempt at a later date to secure certain rights by litigation. The case was finally decided by the Supreme Court of the State, 9th Wyoming Report, 110. This decision is plain. It indicates that the State can regulate the use of water regardless of when the claim was initiated or under what laws water was first used. It is not presumed that the Legislature will injure water users by the passage of any general laws. It is not believed that it will ever be necessary to injure any water user to protect a community. It is essential to have water rights defined in such a way that the individual will never presume to have such rights as will enable him to even threaten the prosperity of a community. If no water user has rights in excess of those that attach to his lands as limited by beneficial use, the community need never fear any trouble. The danger in irrigation matters never begins with the community, or the State. It has its birth in the greed of one or two who, through the weakness of supervision in behalf of the public, are able to get what they are not entitled to, thereby enriching themselves at the expense of their neighbors.

"The fundamental principles which provide equal protection to all

Mr. Johnston. and special privileges to none, are so simple and their number is so small, that it is surprising that they are not stated in every irrigation statute and in every text book dealing with the use of water. Yet we look almost in vain for any discussion of them. All that need be borne in mind is that the right to use water should be limited in accordance with the beneficial use made and the right should belong to that use rather than to the user. All other matters relating to the right are questions of fact that are easily obtained."

Mr. Le Conte. L. J. Le Conte, M. Am. Soc. C. E. (by letter).—Every great hydraulic project calling for a large expenditure of money demands the employment of three distinct classes of Man, namely, a good promoter or business man, a good lawyer, and a good engineer. Each of these men attends to his own particular line of business, and, as a final result, the scheme is a success. This seems to be the experience everywhere.

Of late years, however, the universal knowledge demanded of the engineer has grown to such an extent that, in order to be thoroughly up to date, he has to be well nigh omniscient. The time now seems to be fast approaching when the engineer in charge of a large scheme will be compelled to acquire the necessary knowledge of all three of these classes of men.

The author says that Court decisions are based more on the logic of judges than on the statutes. This calls to mind the lamentable fact that almost everywhere all the law that the general public gets, at best, is judge law, pure and simple.

The author calls on the Engineering Profession to come forward and take active lead in the movement for the enactment of better water laws. This is badly needed everywhere, and certainly engineers are, by experience, better fitted for the task than men of any other class.

The author says that many lawyers fear that the reform in water laws would spoil their business, which is true; but it also shows—a well-known fact—that members of the legal profession, as a rule, make their living by feeding on the misfortunes of the public. It is natural, therefore, that they should object to anything that cuts down the business of the Courts.

It is highly desirable that interstate or national water laws should be framed and enacted as soon as possible, inasmuch as all large projects are now halted and waiting for results. The new State water-right laws of Oregon certainly seem to be simple and effective, and, apparently, they fulfill all public requirements very satisfactorily. It now remains to enact similar laws to cover the needs of interstate streams.

It will be remembered that the City of New York contemplated getting a new water supply from the Housatonic River, which, to a large extent, is in the States of Massachusetts and Connecticut. After

much wrangling in the Courts, the scheme was abandoned, and the city was forced to go to the Catskills, which project is now being developed on a grand scale. Mr. Le Conte.

The report of the Committee on Interstate Water Rights to the National Irrigation Congress, held at Sacramento, Cal., in 1907, seems to be highly commendable. The equitable suppression of the drastic law of riparian rights seems to be absolutely necessary for the public welfare.

Finally, the writer would respectfully suggest, in keeping with what he has stated before, that the Administrative Board, which will have complete jurisdiction of all matters relating to water, should be composed of good engineers, good business men, and good lawyers, all of whom are reputable citizens. The reason for this is that it is now practically impossible to find one class of men gifted with all these qualifications combined.

The members of such a Board, of course, would shoulder grave responsibilities, and their compensation should be ample, so as to enable them to give their time exclusively to the Board's business.

Mr. Dillman. GEORGE L. DILLMAN, M. AM. SOC. C. E. (by letter).—This paper brings up a subject which is most pertinent and timely. If it results in a solution, more good will have attended it than can be put in dollars and cents, and everybody concerned will benefit: the investing public, the constructing companies, the consumer who ultimately pays the bills—everybody but the professional litigant.

Some years ago the writer was employed as engineer for one of two strong litigants on a water-right case in Southern California. The usual slow course in the Courts was followed for years; a final verdict seemed impossible for years to come. Somebody, at a "psychological moment," suggested compromise, and, as far as the legal adjudication of the case is concerned, it will never be settled. After the case was dropped, one of the attorneys was asked: "What can make a good water right in California?" The reply was: "Nobody has any right to such a thing who is not prepared to fight for it at any and all times. There is no such thing as a perfectly defensible water right."

Two strong corporations in Kern County agreed to disagree over the water rights of Kern River, a large stream with many diversions for irrigation purposes. Their case was in the Courts for several years, during which time the small claimants allied themselves with one side or the other. Many thousands of dollars were spent, and the case was withdrawn without a verdict. The principal litigants agreed to divide the whole waters of the river between themselves, and the small fry were shut out. The legality of the dispute is not settled, but both big companies together stand ready to make it interesting

Mr. Dillman. for all adverse claimants. They have acquired adverse claims by prescription now, therefore, there is no question as to their "rights."

In California there are thousands of doubtful rights, involving millions of dollars, which would be settled if it could be done with small or reasonable expense. The owners of new diversions, changes of use, or changes of point of diversion, are not safe in their investments until their rights are prescriptive. Only a few days ago, one of the leading lawyers of California stated that the best water right to be had was by prescription and continual use, that filing rights and riparian rights were so indefinite that their defense was always doubtful.

There will never be much conflict between uses for power and irrigation in California by reason of the physical conditions, provided the power water is returned to the stream after use. In a large majority of cases the power sites are above the irrigation diversions. Where considerable areas of irrigable land lie in valleys above power sites, their irrigation will generally result in the steadier flow of streams below. Where power necessitates storage to increase low-water volume, this storage will benefit the irrigation below, because the irrigation season is generally the time of low water. Therefore irrigation and power uses can be made mutually beneficial.

This same question is international in places. Recently, the papers announced that the Mexican Company had shut the water from California in Imperial Valley. Only a few years ago the United States and Canada came to an agreement over the respective quantities to be diverted at Niagara Falls.

While agreeing with the author on the desirability of definite laws, uniform if possible, respecting water rights, there are some suggestions in the paper which seem wrong.

A National Board, if formed, should certainly have its jurisdiction limited to strictly interstate waters. The difficulty of getting action by any Bureau in Washington hardly warrants the establishment of a new one.

The attitude of the Government officers in connection with water-power control in California is an absolute blockade. They do not allow others to develop it, nor can they develop it themselves. Thus far, conservation has conserved nothing. To have the laws changed so that the Government would develop and supply power at cost would probably be as fatal as the Reclamation Service. In spite of its magnificent publicity department, the Reclamation Service has reclaimed very little in proportion to its expenditure, and only a small portion of that is at such a price that the farmer can afford to pay the rates.

If the Government would grant permits for power development, considering the companies as public service corporations, and regulate

them as completely as they regulate the railroads, the farmer would get his power much cheaper than under Government ownership. Mr. Dillman.

There is a crying need for something definite in water-right law. The experts cannot tell what the law is, and the Court decisions are very contradictory. The engineers of California will all be glad of something that is determinable. The State Commission seems to have made progress in Oregon. Perhaps others will elsewhere. We should welcome an interstate commission if its jurisdiction covered only strictly interstate questions; but, please deliver us from any more Government Bureaus. Those we come in contact with are pernicious.

W. E. MOORE, M. AM. SOC. C. E. (by letter).—Mr. Lewis has contributed a most valuable paper. The need of legislation along these lines has been felt for several years in all the Western States, and will be felt sooner or later in every State in the Union. It is not any exaggeration to state that there is not a single country on the globe which has a water law in accordance with present-day needs. Mr. Moore.

Oregon certainly has the best law governing water rights of which the writer has any knowledge. It is a long step in the right direction. With its general outline he agrees most heartily, but thinks it does not go far enough—does not cover as much ground as it should. In some respects it should be more specific, for example, in defining the duties of the engineer in granting a right on a stream before he becomes thoroughly familiar with that stream. It requires several years to learn all the characteristics of any stream, and the writer is opposed to any one granting a water right unless the water is there to fulfill the grant. It is true that, in case of a shortage, the Oregon law shuts off the latest appropriators, but these appropriators ought to know whether they will get water for a part of every year, and what part. As every one knows, the low-water flow of every stream varies from year to year, and such data should be available in the office of every State Engineer in order that the water users of that stream will know on what to depend.

There is a growing belief that every State law should limit the water in every case to the least quantity necessary. This is desirable for various reasons; it admits of a wider use of the waters, and, consequently, more water users, and it tends to prevent waste which sooner or later becomes a menace to the public health.

In the writer's opinion, every water contract should be submitted to the State Engineer for approval. In this way many of the contentions between unscrupulous promoters and *bona fide* water users will be avoided. He would also make all plans for the construction of hydraulic works subject to the approval of the State Engineer. There has been a great deal of speculating on water rights granted by State Engineers in several of the Western States, when in reality

Mr. Moore. the grants amounted to nothing. The time has come when such a thing should be impossible. When the State Engineer grants a water right for any purpose, investing capital should be able to rest assured that the water is there, and that the system or plant by which it is to be used will be constructed along right lines. This is just as essential for the water user as it is for investing capital, for it is the water user who ultimately pays the bills, and the better he is protected in his rights, the better will it be for the lasting prosperity of the community.

That Federal legislation is needed on interstate streams is certainly patent to every one. The problem cannot be handled equitably by the different States until the nature of the human race is radically changed. These waters can be apportioned properly between the interested States by the National Government, and that certainly ought to be done without delay, as the problem becomes more complicated from year to year and retards progress very materially.

The writer certainly does not agree with Mr. Lewis in advocating the construction and operation of power-plants by either the Federal or State Governments, unless such work can be entirely and absolutely eliminated from politics.

One of the most valuable features in the operation of the Oregon law is shown by the fact that during the three years it has been in force, it has clearly demonstrated that practically all the contentions over water rights can be satisfactorily settled outside of the Courts. Even the legal profession admits that legal practice has become entirely too complicated and cumbersome, thereby causing injurious and frequently fatal delay. It is obvious, therefore, that anything that will hasten the settlement of contentions and lessen their cost will be a benefit to the community. The writer has no desire to deprive the legal profession of anything that rightfully belongs to it, but it is just as reasonable to contend that the engineer could settle legal questions as it is to argue that the lawyer could settle engineering questions. It is a fact that many engineering questions are intimately connected with legal questions and cannot be separated from them, but as far as possible they should be kept apart. To the writer's mind this is one of the most important problems with which the Engineering Profession has to deal at the present time, and he feels very grateful to Oregon for the many valuable lessons it has given us.

Mr. Bien. MORRIS BIEN, ESQ.* (by letter).—This paper is very valuable in summarizing the present situation regarding water rights in the Western States, and will enable both engineers and lawyers to grasp more fully the great importance of the problems which are yet to be solved in regard to the determination of rights to the use of water.

* Supervising Engineer, United States Reclamation Service.

In a country where most of the land is practically valueless unless it can receive a water supply for irrigation, it seems strange that it should take so long to provide for a satisfactory and conclusive record of water titles comparable with the record of land titles. The first step in this direction was taken by the State of Wyoming in 1890, and some of the other arid land States have followed along the general lines adopted there. Mr. Bien.

Large investments, running well into the millions, in connection with irrigation and water-power construction in the arid regions, depend fully as much on the title to the use of water required as on the title to the land whereon the structures are built.

Many enterprises of considerable importance have found themselves compelled to defend in the Courts their right to the use of water essential to the enterprise, the usual examination of the record in such cases giving practically no indication regarding the true condition of the water supply.

So long as it is possible in a number of the arid States for any one to place on record a claim to a quantity of water to be diverted from a particular stream far in excess of its flood flow, and entirely regardless of the fact that all or a large part of the available water supply may have been put to beneficial use, just so long will diastrous failures of water supply be encountered.

Even the Courts of some of the Western States have so far placed faith in such records as to issue decrees declaring water rights vested in litigants which could call for a water supply of many times the greatest known discharge.

The modern system of water law designates the State Engineer as an expert witness to determine all the features necessary for fixing the extent of the vested water rights and the quantity of unappropriated water which may be available for future enterprises. His findings are conclusive, if not appealed to the Courts, and, when thus appealed, his expert determination of the essential facts would not be disturbed except on proof of error.

This is radically different from the bewildering and contradictory statements by ill-qualified witnesses regarding flow of water, capacity of ditches, areas irrigated, etc., which even now are characteristic of litigation regarding water rights in a number of the Western States.

The system which is now being worked out in Oregon and in some of the other Western States will ultimately provide a record of titles to the use of water as reliable within the necessary limitations as our records of title to land under the most up-to-date systems, substantially equivalent to the Torrens system of land title records. The right to the use of water differs from a fee simple title to land in two essential particulars: first, there is no ownership of the *corpus* of the water as there is of the land, the right in the former case is only a right of

Mr. Bien. use; second, the water of the streams is not fixed, but transitory, and, moreover, the quantity available fluctuates from day to day and year to year. Nevertheless, it is possible to provide for an adjustment of the respective rights to the use of the water, and to make such rights as easily determinable as the ownership of land.

The questions of transfer of the place of use and changes in the method of diversion are of extreme difficulty, and before transfer or changes are permitted without loss of priority careful expert investigation is essential.

In one case a considerable area of land bordering on a small stream with the appurtenant water rights was purchased and the place of use and method of diversion were radically changed. Instead of small ditches diverting the water short distances from the stream and after irrigation allowing return seepage to the stream, the entire quantity of water thus claimed was diverted in an iron pipe and carried to another water-shed. It is claimed by water users below this point of diversion that they have suffered a serious diminution in the quantity of water which had formerly been used on the land above and returned to the stream to become available for their use. They claim that the purchaser of the upper lands could not take a large proportion of the water out of the drainage area without seriously impairing their rights. It will doubtless require a decision of the Courts to settle this question.

The determination of water rights on interstate streams is one which, at an early date, should receive the attention of the Federal and State Governments. The plan suggested by Mr. Lewis is undoubtedly the only logical solution of the problem, and these two Governments must co-operate in working out its details.

The principle that water rights on interstate streams must be adjudicated independently of State lines, while indicated in a general way by the decisions of the United States Supreme Court, is not wholly settled.

In the case of Kansas *v.* Colorado, the Supreme Court took the first step, but without definitely announcing such a principle. In the case of Bean *v.* Morris (221 U. S., 485), the Supreme Court assumed that the States would recognize prior vested rights in another State affecting a stream common to both States.

The question, however, is not decided definitely by these cases, and it may be that the United States Supreme Court will lay down this rule definitely in the case recently begun in that Court by the State of Wyoming against the State of Colorado which seems to rest almost wholly on this question.

Mr. Sheley. HORACE W. SHELEY, Assoc. M. AM. SOC. C. E. (by letter).—For the sake of clearness, the writer has divided his discussion of this timely paper into five parts. Although he has made special mention

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of irrigation rights, he believes, with Mr. Lewis, that the same principles can be applied to the use of water for other purposes.

1.—Measurement of Water.—Before it will be possible to make water rights definite, certain modifications in the units of measurement must be made. Most of the Western States have already abandoned the variable "miner's inch" for the cubic foot of water per second, commonly called the second-foot, but this is a rate of flow and not a quantity; this term conveys no more meaning than the answer "sixty miles an hour" would give to a question about the distance by railroad between two cities. So far as the writer knows, Nevada is the only State which has named a quantity of water, instead of a rate, when fixing water rights. In that State the unit of measure is the acre-foot. Some other States mention the acre-foot in the regulations of their State Engineer's Offices, but do not give it in their statutes.

A further step remains, namely, to fix the maximum rate at which a given quantity of water may be taken; this corresponds to the "peak load" of electrical engineers.

2.—Place of Measurement.—For new irrigation enterprises it is generally better to name some point near the place of use as the point of measurement, in retailing water, because then the wholesaler, or constructor of the irrigation works, cannot be accused of not having properly constructed his reservoirs and canals so as to prevent seepage and waste in transit. If this method had been followed by a certain large company in one of the arid States, it would not now be in legal difficulties over the failure of the floor of a reservoir to hold water, because it could deliver the requisite quantity from another reservoir.

Measurement at the place of use makes it possible for a later comer to improve the ditch of the old appropriator, and take the water formerly wasted in transit for use on his own land. It is an incentive to the older appropriator to make these betterments himself, lest some one else do it and take this water from him.

3.—Changes in Place or Manner of Use.—Because of his experience, both in Utah, where changes in the place and manner of use are permissible, and in adjoining States, where the water is appurtenant to the land, the writer believes that the Utah policy is the better, leading to higher duties and more economical use of the water.

If a farmer has more water than he needs for a given tract, as a result of an excessive appropriation in the beginning, of a change in crops, of more thorough cultivation, or of a rise in ground-water, he will continue to use all the water if he cannot transfer all or part of it to other land without loss of priority. If the law permits the transfer under proper safeguards to others, it leads to greater duty for the water.

Mr. Sheley.

On the other hand, if the water is appurtenant to the land, so that the owner cannot transfer any part of it, it will be necessary for any one desiring to use it to go to the Courts or to a special tribunal, in order to do so, and the difficult burden of proving that the present owner is not applying the water to the best advantage will be on the new comer.

It is sometimes desirable *pro bono publico* to change the manner of use. The laws of Wyoming recognize that some uses are more important than others, and state the preferences, as follows:

"First—Water for drinking purposes for both man and beast. Second—Water for municipal purposes. Third—Water for the use of steam engines and for general railway use. Fourth—Water for culinary, laundry, bathing, refrigerating (including the manufacture of ice), and for steam and hot-water heating plants. The use of water for irrigation shall be superior and preferred to any use where turbines or impulse water-wheels are installed for power purposes."

4.—Adjudication of Water Rights.—In 1908 the writer assisted the Hon. Caleb Tanner, State Engineer of Utah, in the tabulation of the statements of 1 200 or 1 300 claimants of water rights in the Weber River water-shed, and since then he has had small respect for the truthfulness and reliability of the claims of owners of water rights. Most of the claims were exorbitant. Our Courts have not been averse to granting absurd claims, and in one instance a judge decreed more than 14 ft. depth as the necessary quantity for a tract of land, where 3 ft. would have been ample.

The system of adjudication in Utah has failed completely, thus far, through lack of money to complete the adjudications and the inherent defects in the method used there. The surveys for the adjudication of the Weber River system were started in 1903 or 1904, but the matter has not even reached the District Courts.

The writer believes that the system first used in Wyoming and later modified and used in Oregon, is the only one that is practical.

5.—Interstate Rights.—The position taken by Mr. Morris Bien in the quotation given by Mr. Lewis, to the effect that the States have not now, and never have had, a right to control the waters within their boundaries, seems to be extraordinary, in view of the fact that the United States Government permitted the Territorial Legislatures to make water laws, and it has applied to the States for water rights, as if it were a private corporation, when undertaking irrigation enterprises under the Reclamation Act. Possibly the context of Mr. Bien's statements would explain the paragraph quoted.

All who have had any experience with the red tape and delays at Washington will hesitate about surrendering any State rights to it, unless absolutely necessary.

In conclusion, the writer agrees with others who are studying water laws and the use of water, believing that the principles of

preferential use, beneficial use, and priority can and will ultimately be adopted in the determination of rights both in one State and between States. Mr. Sheley.

Morris Knowles, M. Am. Soc. C. E.—The speaker is particularly grateful to the author for his clear exposition of the doctrine of appropriation as practiced in Oregon; and is personally interested in the discussion of this important question, as he has the honor of being President of an Association, one important object of which is the improvement and rationalization of the water laws of Pennsylvania. The paramount importance, at this time, of flood prevention and protection, power development, water supply, improvement of navigable rivers, and the whole broad subject of water conservation in every part of the Union, points to the necessity of adopting in every State a well-considered, comprehensive plan, based on adequate State and National legislation. Mr. Knowles.

The speaker agrees most heartily with the suggestion that the Engineering Profession generally, and engineering societies particularly, not only can, but ought to lead in the discussion of every phase of the subject, and lend their aid to the framing of legislation on such a matter, for the consideration of which their members are peculiarly fitted, in many respects better than any other class of citizens. It is time we were putting into action in this field the policy urged by President Ockerson at the Seattle Convention.

Prior Appropriation vs. Riparian Law.—On one point, however, the speaker holds an opposite opinion from the author, which illustrates the peculiar situations that may arise when engineers undertake the discussion of the intricacies of the law.

The case of Kansas *v.* Colorado did not, in the speaker's opinion, declare that "the doctrine of riparian rights, * * * is not the law, and therefore never has been the law," even of interstate waters. On the contrary, Kansas *v.* Colorado definitely followed the principles of riparian rights, as between Kansas and Colorado. The fact that the Colorado use in question was on non-riparian lands did not affect the issue. The Court considered the decision from the point of view of the rights of Kansas as a State against those of Colorado as a State, and not of the rights of individual riparian proprietors in Colorado. These individual rights come under the rule previously enunciated in Anderson *v.* Bassman (140 Fed., 22) and later affirmed by the Supreme Court in Rickey, etc., Co. *v.* Miller (218 U. S., 258) that the riparian owner deduces his rights from the law of his own State and that the private right is "not in his own right, but by reason of and subordinate to the rights of his State" (Turley *v.* Farman, 114 Pac., 278). In the case of Rickey, etc., Co. *v.* Miller, which involved the rights of riparian owners in California and appropriators in Nevada, the rule followed was that

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"the enforcement of either right beyond the boundary of its State must depend upon the concurrence of the other State. Unless the upper State (California) will voluntarily impose conditions upon its citizens in favor of users in the lower State (Nevada) the latter have no right in the matter other than to complain that the lower State as such (and not merely the plaintiff) is not receiving an equitable share of the benefit of the stream."*

The language of the Court in Kansas *v.* Colorado makes plain that the principle of equal rights, which is the basis of riparian rights, was followed. For example, referring to the law in Kansas, which is the law of riparian rights, Justice Brewer said:

"As Kansas thus recognizes the right of appropriating the water of a stream for the purpose of irrigation, subject to the condition of an equitable division between the riparian proprietors, she can not complain if the same rule is administered as between herself and a sister State."

(The use of the word "appropriating" here appears to the speaker to be somewhat unfortunate, but need cause no confusion, for it bears no connection whatever with the doctrine of "appropriation" as practiced in other Western States. Apparently, the word "diverting" would have expressed the idea of the learned judge without danger of confusion.) And again, referring to the statement quoted by the author, from the conclusion of the Court, in the Kansas *v.* Colorado case, "* * * it is obvious that if the depletion of the waters of the rivers by Colorado continues to increase, there will come a time when Kansas may justly say that there is no longer an equitable division of benefits, and may rightfully call for relief against the action of Colorado."

No support is given by this language to the theory that there may exist here any exclusive right by priority. This is the language of the riparian rights doctrine.

Then, since the premise is in error, the conclusions that "riparian rights are not considered as vested rights by the Supreme Court," and that "if Congress should decide that the enforcement of the doctrine of priority of appropriation and beneficial use, * * * should constitute an equitable apportionment of benefits, it is conceivable that the Supreme Court would uphold such act," must fall. In the speaker's opinion, such an act, in the light of the present generally accepted views of the powers of Congress, would be unconstitutional, both as outside the enumerated powers of the Federal Government, and as an invasion of the right of private property. In fact, the Kansas *v.* Colorado decision says:

"It [each State] may determine for itself whether the common law rule in respect to riparian rights, or that doctrine which obtains in the arid regions of the West of the appropriation of water for the purposes of irrigation shall control. Congress cannot enforce either rule upon any State."

* Wiel, "Water Rights in the Western States," 3d ed., p. 364.

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We of the East and the country at large must look to other measures than the adoption of the law of appropriation in its entirety for the development and conservation of our water resources.

Executive Board vs. Court Administration.—On the other hand, the speaker agrees with the author in believing that water laws may be better administered by executive boards than by the Courts and by injunction; and he believes that it might be not only lawful, but very serviceable, for Congress to provide the machinery for administering the law with respect to interstate waters, provided the law can be said to be already determined by Court decision. This, however, does not give ground for the broad assertion that "Congress must have power to prescribe by law what shall constitute an equitable division of benefits as to interstate waters." In fact, as already stated, it would seem plain from Kansas *v.* Colorado that Congress does not have that power—a very different one from the power to provide the machinery for administering the law after the decisions of the Supreme Court have prescribed what shall constitute an equitable division of benefits.

This does not in any sense mean that the speaker does not believe that the Federal Government, under the Constitution, can co-operate with the States in exercising its control over the navigability of streams, and their tributaries also, in such a way as to secure great collateral benefits. On the contrary, he holds that opinion most strongly, and believes in the propriety of the adoption by Congress of the Newlands River Regulation Bill and similar measures.

There may be a question, also, as to whether the law has become sufficiently determined to justify the creation of an administrative commission. The doctrine in Kansas *v.* Colorado was clearly stated; but whether this would apply under all conditions is rendered somewhat doubtful by the following from the decision in Rickey, etc., Co. *v.* Miller (218 U. S., 258, 261).

"It is conceivable, to be sure, that the decisions of this Court may determine that the States have rights as against each other *in invitum* in streams that flow through the land of both. (Kansas *v.* Colorado, 206 U. S., 46, 84; Mo. *v.* Ill., 200 U. S., 496, 519, 520.) The rights may vary according to the system of law required by natural conditions. They may be more or less analogous to common law rights between upper and lower proprietors, where irrigation is not necessary, as in most of the older States. (See N. Y. *v.* Pine, 185 U. S., 93, 96.) There may be some, perhaps limited, right of appropriation in the upper State, at least in the water-shed of the stream, where irrigation is the condition of using the land. (See Kas. *v.* Col., 206 U. S., 46, 100-104, 117.) But whatever this Court may decide, if a private owner should derive advantage from such a decision, it would not be in his own right, but by reason of and subordinate to the rights of his State. * * *"

If it be true that the law for all cases has not been determined, it would appear almost certain that any important contest between States

Mr. Knowles. before an administrative board would be appealed to the Supreme Court, until the universal law had become established, and a Commission might therefore be of no value at present. In addition, it is debatable whether enough important contests between States will arise to require the continuous service of a Commission.

State Commission Administration.—None of these objections applies, however, to administration of intra-state water laws, on a basis of equitable apportionment, by State Commissions. States undoubtedly have power to determine by law what shall constitute an equitable apportionment of the use of water within their borders; and the delegation of this power to a commission cannot be opposed on the ground that it must be exercised only by direct action of the Legislature, any more than the regulation of public utilities by Commission, now so firmly established, can be so attacked. The words of Justice Timilin in Minneapolis, etc., Railway Co. *v.* Wisconsin Railroad Commission, are applicable:

"It is argued that the power to fix rates is a legislative one and can never be anything else; * * * that the legislative power is by the Constitution vested in the Senate and Assembly, and cannot be set apart except as expressly provided for in the Constitution; but when we add to this that, because of the multitude of detail, the intricacy of the subject, the expert knowledge required, the numerous separate investigations of inter-related questions of fact which are necessary * * * a legislative body * * * would find it an actual rather than a legal impossibility to fix just and reasonable rates, it becomes apparent that this position tends to the conclusion that the State * * * was shorn of some of its usual and necessary power of sovereignty and became impotent to exercise the power of regulation. Regulation by direct action of the legislature has been tried and found impracticable and its attempt generally abandoned."

The speaker has not yet attempted to work out the details of such an administrative riparian system, but believes that it may be possible, by making simple and rapid the determination of riparian rights and of the terms of an equitable division by a suitably constituted commission, and by making proper provision for the condemnation of such rights for public uses under the supervision of such a board, to develop a system which will lead to a full utilization of the waters of a State, with ample protection both to the public and the investor, and with a possibility of obtaining in some instances great collateral public benefits in the way of flood protection, development of water power, improvement of navigation, improvement of quality of water, etc.

The feasibility of such a scheme appealed to Wiel, who says:*

"This system of law would seem to offer a field for administrative legislation; in fact, a readier field than the law of prior appropriation. Where the test is what is reasonable in each case, discretion must necessarily come into play, whereas where parties have exclusive rights

* "Water Rights in the Western States," 3d ed., p. 830.

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measured by priority there is * * * little room for the exercise of discretion by administrative officers * * *. Where the common law applies the test of reasonableness, legislation is apt and readily applied; as, for example, in dealing with public service companies. The common law says their rates and regulations must be 'reasonable,' and accordingly public service commissions and similar bodies are created. Likewise under the new law of percolating water 'reasonable use' has become the test, and statutory regulation based thereon is being adopted. As yet, however, there has been no attempt to provide a statutory system governing the reasonable use of water by riparian proprietors among themselves, in jurisdictions applying that system, though there would seem a clear field for such legislation if desired."

Limit of Appropriation by Reasonable Use.—The speaker would like to call attention also to the unmistakable tendency (illustrated by the "Pro-rating" statute of Washington, the constitutional limitation of the right to appropriate in Idaho, and such decisions as Basey *v.* Gallagher, 87 U. S., 670; Union Mining Co. *v.* Dangberg, 81 Fed., 73; Anderson *v.* Bassman, 140 Fed., 14; and Schodde *v.* Twin Falls L. & W. Co., 161 Fed., 43) to depart from the strict law of prior appropriation, and to limit appropriation by a requirement of reasonableness, which is doing much to narrow the gulf between the doctrines of appropriation and riparian rights. It appears to him that, as density of population increases in the Far West, this tendency will increase; and that uniformity in the administration of State laws will be approached, in spite of differences in form, by the application of the "rule of reason" to both systems of law, rather than by the abandonment of either one in favor of the other.

On this phase of the subject, Mr. Morris Bien, Supervising Engineer of the U. S. Reclamation Service, speaking before the National Irrigation Congress at Spokane, in 1909, said:

"The doctrine of rights by prior appropriation has been adopted in nearly all the States where irrigation is required; but this doctrine as now generally understood will necessarily require modification. * * *

"In the Yale Law Journal for January, 1909, is a discussion of the idea of reasonable use, whether under the doctrine of riparian rights or the doctrine of appropriation. It shows that the courts have frequently called attention to the fact that the doctrine of appropriation must be modified by the idea of reasonable use which is also a fundamental limitation of the riparian doctrine. This idea of reasonable use will undoubtedly become an important factor in future years when valuable interests depending upon the entire water supply have grown up within many of the irrigation districts, and it becomes necessary to protect these interests in cases of temporary deficiencies which sometimes continue for a number of years in succession. * * * The qualification of the doctrine of prior appropriation by the idea of reasonable use, and the application of the same idea to the riparian doctrine will undoubtedly bring these opposing doctrines much closer together in

Mr. Knowles. actual practice, and is likely in the end to cause a practical uniformity in the governing principles of all the irrigation States."

Best Development of Water Resources.—In conclusion, the speaker wishes to raise the question whether the "Water Power Policy" advocated by the author is necessary to attain the desired ends. If so, the speaker would agree that "the States, in co-operation with the United States," should "develop this power and supply at cost plus interest." But, if it is possible to attain the same ends in other ways and without the tedious delays that must precede such a consummation, the speaker does not see the necessity of adding further commercial enterprises to the burdens of our State Governments. In the belief that, under a rational, well-defined system of water laws, and with wise State regulation in the interest of the people, private capital would construct the works necessary to the conservation of water, the Pennsylvania Water Conservation Association has been formed. This Association, including in its membership capitalists, publicists, civic bodies, power companies, and water companies, has for its object the formulation of a plan for the development and utilization of the water resources of the State, by means of private capital, under improved laws and State supervision, in such a way as to offer a safe, attractive field for investment; to insure reasonableness of rates and safety of construction; to secure, wherever possible, by supervision of designs and operation, prevention of floods and improvement of rivers; and to serve by a broad, far-sighted policy of conservation the public interests of this and of future generations.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

A SHORTENED METHOD IN ARCH COMPUTATION.

Discussion.*

By William Cain, M. Am. Soc. C. E.

William Cain, M. Am. Soc. C. E. (by letter).—The author has made a successful attempt to shorten certain calculations pertaining to the theory of the arch without hinges, where single loads are considered. As the number of parts into which the arch ring is divided is increased, the burden of computation is very much increased, and any device for shortening the work will be appreciated by the computer. Mr. Cain.

As the subject is of such practical importance, the writer will give a method of computing the quantities in question by a brief and independent procedure, and will incidentally derive check formulas for the difference method proposed by the author.

In the diagram, Fig. 2, the horizontal distances of the loads, P_1, P_2, . . . , from the center of the span, A, will be denoted by d_1, d_2, ...

From any P, as P_2, are drawn the two sides of the trial equilibrium polygon pertaining to this load, the one to the right of the load, $P_2\,A$, being horizontal, the one to the left, $P_2\,B_2$, being inclined at an angle of 45° to the horizontal. It follows that $b_4\,h_4 = h_4\,P_4$, $b_4\,h_3 = h_3\,P_4$, etc.; whence, for the lines, $P_4\,B_4$, $P_3\,B_3$, we have,

$$\begin{aligned} b_4\,h_4 &= z_4 - d_4 \\ b_4\,h_3 &= z_3 - d_4 \\ b_4\,h_2 &= z_2 - d_4 \\ b_4\,h_1 &= z_1 - d_4 \\ \hline \sum_o^4 b_4\,h &= \sum_o^4 (z) - 4\,d_4, \end{aligned} \qquad \begin{aligned} b_3\,h_3 &= z_3 - d_3 \\ b_3\,h_2 &= z_2 - d_3 \\ b_3\,h_1 &= z_1 - d_3\,, \\ \hline \sum_o^3 b_3\,h &= \sum_o^3 (z) - 3\,d_3. \end{aligned}$$

* This discussion (of the paper by H. A. Sewell, Esq., published in October, 1912, *Proceedings*, but not presented at any meeting), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Cain.

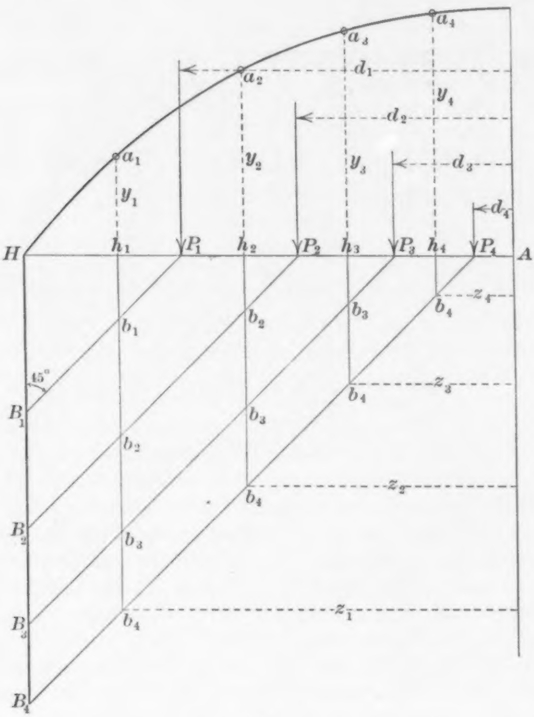

FIG. 2.

Or, more generally, dropping the b subscripts, the sums can be indicated thus:

$$\sum_o^n (bh) = \sum_o^n (z) - n\, d_n \quad \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

Thus, for $n = 4$, referring to the load, P_4, and the side of its equilibrium polygon, $P_4\ B_4$,

$$\sum_o^4 (bh) = b_4\, h_4 + b_4\, h_3 + b_4\, h_2 + b_4\, h_1; \qquad \sum_o^4 (z) = z_1 + z_2 + z_3 + z_4.$$

This formula enables us to compute directly $\sum_o^n (bh)$ corresponding to $P_n\ B_n$ for any P_n, or for $n = 1, 2, 3, \ldots$ in turn.

Now let,

$$\sum_o^n (bh.z) = bh_1.z_1 + bh_2.z_2 + bh_3.z_3 + \ldots + bh_n.z_n.$$

Thus, if $n = 4$, the bh's refer to the line $P_4\ B_4$. It follows, from the relations given above, that, Mr. Cain.

$$\sum_o^n (bh.z) = (z_1 - d_n)\, z_1 + (z_2 - d_n)\, z_2 + \ . \ . \ . + (z_n - d_n)\, z_n$$
$$= (z_1^2 + z_2^2 + \ . \ . \ . + z_n^2) - d_n\, (z_1 + z_2 + \ . \ . \ . + z_n)$$

or,

$$\sum_o^n (bh.z) = \sum_o^n (z^2) - d_n \sum_o^n (z) \qquad \ldots\ldots\ldots\ldots (2)$$

in which the notation is sufficiently explained by the equivalent expansions above. Similarly, if,

$$\sum_o^n (bh.y) = bh_1.y_1 + bh_2.y_2 + \ . \ . \ . + bh_n.y_n,$$

indicates the sum of quantities of the type $(bh.y)$ for any line $P_n\ B_n$ corresponding to a load, P_n, we have,

$$\sum_o^n (bh.y) = (z_1 - d_n)\, y_1 + (z_2 - d_n)\, y_2 + \ . \ . \ . + (z_n - d_n)\, y_n$$
$$= (z_1 y_1 + z_2 y_2 + \ . \ . \ . + z_n y_n) - d_n\, (y_1 + y_2 + \ . \ . \ . + y_n)$$

which can be indicated by the shorter notation,

$$\sum_o^n (bh.y) = \sum_o^n (z\, y) - d_n \sum_o^n (y) \qquad \ldots\ldots\ldots\ldots (3)$$

Thus the sums desired, for P_n and its corresponding $P_n\ B_n$, can be computed directly from Formulas (1), (2), and (3), on giving the proper numerical value to n.

Before giving a numerical application of the formulas, it may be well to derive the author's difference formulas from them.

Thus, if in Formula (1), we change n to $(n - 1)$ and subtract, we find,

$$\sum_o^n (bh) - \sum_o^{n-1} (bh) = z_n - n d_n + (n - 1)\, d_{n-1}$$
$$= (n - 1)\, (d_{n-1} - d_n) + (z_n - d_n) \qquad \ldots\ldots (4)$$

This formula can also be obtained directly from the figure. Thus, take $n = 4$, $n - 1 = 3$, then the left number of Formula (4) equals $3\ (b_4\ b_3) + b_4\ h_4$, which reduces to $3\ (d_3 - d_4) + (z_4 - d_4)$, or to the right member.

Similarly, in Formula (2), change n to $(n - 1)$ and subtract. Therefore,

$$\sum_o^n (bh.z) - \sum_o^{n-1} (bh.z) = z_n^2 + d_{n-1} \sum_o^{n-1} (z) - d_n \sum_o^n (z)$$
$$= z_n^2 + d_{n-1} \sum_o^{n-1} (z) - d_n \left(\sum_o^{n-1} (z) + z_n \right)$$
$$= (d_{n-1} - d_n) \sum_o^{n-1} (z) + (z_n - d_n)\, z_n \qquad \ldots\ldots\ldots\ldots (5)$$

Mr. Cain. A similar procedure, using Formula (3), gives,

$$\sum_{o}^{n} (bh.y) - \sum_{o}^{n-1} (bh.y) = z_n\, y_n - d_n \sum_{o}^{n} (y) + d_{n-1} \sum_{o}^{n-1}$$

$$= (d_{n-1} - d_n) \sum_{o}^{n-1} (y) + (z_n - d_n)\, y_n \text{.........} \quad (6)$$

Formulas (5) and (6) can likewise be derived directly from the figure by developing the left members and reducing. It will be observed that Formulas (4), (5), and (6), are equivalent to the author's formulas on noting that $d_n = \frac{L}{2} - p_n$.

To illustrate the application of Formulas (1), (2), and (3), take the segmental arch considered in the writer's "Theory of Solid and Braced Elastic Arches,"* where only eight divisions of the semi-arch were made. Of course, in a practical design, a greater number of divisions are essential for fairly accurate results, so that this particular investigation is only intended to illustrate the method for any arch, using any number of dimensions.

The neutral line of the arch is shown in Fig. 3, also the two sides of the equilibrium polygon for each load P_n ($n = 1, 2, 3, \ldots$) are drawn from the computations in the book.

To effect these computations, the sums, $\Sigma\,(bh)$, $\Sigma\,(bh.z)$, $\Sigma\,(bh.y)$, are needed, and these will now be found for each load by aid of the writer's Formulas (1), (2), and (3), which will thus show the great saving in the labor of computation over the method used in the text.

The values of the vertical ordinates, $y_1, y_2, \ldots$, at the points, $a_1, a_2, \ldots$, the horizontal distances, $z_1, z_2, \ldots$, of these same points from the center of the span, and the horizontal distances, $d_1, d_2, \ldots$, of the unit loads, $P_1, P_2, \ldots$, from the center of the span, were all measured from a large-scale drawing, and their numerical values are all inserted in Tables 2, 3, and 4, together with certain derived sums needed in the application of the formulas. The method of computation is sufficiently indicated in Tables 2 and 3. In Columns 16, 18, and 20, of Table 3 are found $\Sigma\,(bh)$, $\Sigma\,(bh.z)$, $\Sigma\,(bh.y)$, for the trial equilibrium polygons corresponding to each load, $P_1, P_2, \ldots, P_8$ in turn. From these derived quantities, by use of very simple formulas given in the text quoted, the values of the horizontal thrust, the vertical components of the reactions at the springings, and the ordinates of the equilibrium polygons at the loads and at the springings are quickly found for each load, P. The equilibrium polygons are now to be drawn, as shown in Fig. 3, and the arch investigated for the actual live and dead loads.

In Table 4, the quantities, $\Sigma\,(bh)$, $\Sigma\,(bh.z)$, $\Sigma\,(bh.y)$, are computed by the author's method of differences, using Formulas (4), (5), and

* Chapter V, Second Edition.

(6). Some care must be exercised here to avoid mistake. Thus, suppose the quantities pertaining to P_3 are to be computed. Substitute $n = 3$ in each of the formulas at the tops of the columns and put the results in the same line with P_3. Then add the results in Columns (4) and (5) on the same line with P_3 to the previous $\sum_o^2 (bh) = 6.05$ to get $\sum_o^3 (bh) = 10.46$, as given in Column (6). Similarly, proceed with Columns (8) and (9) to find $\sum_o^3 (bh.z) = 117.01$, as given in Column (10), and with Columns (12) and (13) to find $\sum_o^3 (bh.y) = 40.47$, as given in Column (14). Mr. Cain.

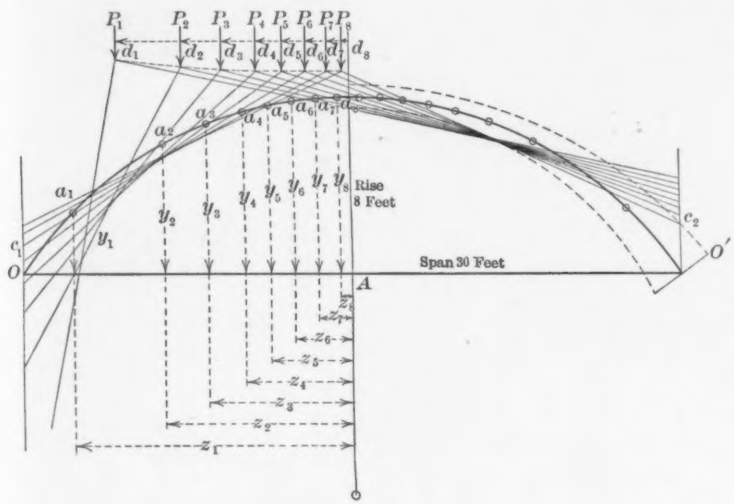

FIG. 3.

With a little care, it is hardly necessary to repeat the numbers added, so that the table can be compressed to about one-third the space here used. As a complete check on the results, the values given for $n = 8$ in Columns (6), (10), and (14), can also be found by use of Formulas (1), (2), and (3), exactly as indicated in Table 3, where n is given the value 8. Further, a check may be given where any value of n, as $n = 4$, is reached, by the use of the same Formulas (1), (2), and (3). The independent method illustrated in Table 3 is somewhat shorter than the difference method, but it can only be checked by repeating the computations. This complete check on the work is the principal gain afforded by the author's method of dif-

Mr. Cain.

TABLE 2.

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
n	Point.	y	$\Sigma_0^n (y)$	y^2	z	$\Sigma_0^n (z)$	z^2	$\Sigma_0^n (z^2)$	$z\,y$	$\Sigma_0^n (z\,y)$
1	a_1	2.77	2.77	7.67	12.72	12.72	161.80	161.80	35.23	35.23
2	a_2	5.74	8.51	32.95	8.73	21.45	76.21	238.01	50.11	85.34
3	a_3	6.72	15.23	45.16	6.65	28.10	44.22	282.23	44.69	130.03
4	a_4	7.28	22.51	53.00	5.08	33.18	25.81	308.04	36.98	167.01
5	a_5	7.60	30.11	57.76	3.79	36.97	14.36	322.40	28.80	195.81
6	a_6	7.81	37.92	61.00	2.63	39.60	6.92	329.32	20.54	216.35
7	a_7	7.92	45.84	62.73	1.53	41.13	2.34	331.66	12.12	228.47
8	a_8	7.99	53.83	63.84	0.51	41.64	0.26	331.92	4.07	232.54
		53.83		384.11	41.64		331.92			

TABLE 3.

(12)	(13)	(14)	(15)	(16) = (7) — (15)	(17)	(18) = (9) — (17)	(19)	(20) = (11) — (19)
n	Load.	d_n	nd_n	$\Sigma_0^n (bh) = \Sigma_0^n (z) - n\,d_n$	$d_n \; \Sigma_0^n (z)$	$\Sigma_0^n (bh.z) = \Sigma_0^n (z^2) - d_n \; \Sigma_0^n (z)$	$d_n \; \Sigma_0^n (y)$	$\Sigma_0^n (bh.y) = \Sigma_0^n (z\,y) - d_n \; \Sigma_0^n (y)$
1	P_1	10.73	10.73	1.99	136.49	25.31	29.72	5.51
2	P_2	7.70	15.40	6.05	165.16	72.85	65.53	19.81
3	P_3	5.88	17.64	10.46	165.23	117.00	89.55	40.48
4	P_4	4.43	17.72	15.46	146.99	161.05	99.72	67.29
5	P_5	3.22	16.10	20.87	119.04	203.36	96.95	98.86
6	P_6	2.09	12 54	27.06	82.76	246.56	79.25	137.10
7	P_7	1.02	7.14	33.99	41.95	289.71	46.76	181.71
8	P_8	0.24	1.92	39.72	9.99	321.93	12.92	219.62

Mr. Cain.

TABLE 4.

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
n	Load.	$d_{n-1} - d_n$	$(n-1)(d_{n-1} - d_n)$	$z_n - d_n$	$\Sigma_o^n (bh) = \Sigma_o^{n-1} (bh) + (4) + (5)$	$\Sigma_o^{n-1} (z)$	$(d_{n-1} - d_n) \Sigma_o^{n-1} (z)$	$(z_n - d_n) z_n$	$\Sigma_o^n (bh.z) = \Sigma_o^{n-1} (bh.z) + (8) + (9)$	$\Sigma_o^{n-1} (y)$	$(d_{n-1} - d_n) \Sigma^{n-1} (y)$	$(z_n - d_n) y_n$	$\Sigma_o^n (bh.y) = \Sigma_o^{n-1} (bh.y) + (12) + (13)$
1...	P_1		0	1.99	1.99 3.03 1.03	0	0	25.32	25.32 38.54 8.99	0	0	5.51	5.51 8.39 5.91
2...	P_2	3.03	3.03	1.03	6.05 3.64 0.77	12.72	38.54	8.99	72.85 39.04 5.12	2.77	8.39	5.91	19.81 15.49 5.17
3...	P_3	1.82	3.64	0.77	10.46 4.35 0.65	21.45	39.04	5.12	117.01 40.74 3.30	8.51	15.49	5.17	40.47 22.08 4.73
4...	P_4	1.45	4.35	0.65	15.46 4.84 0.57	28.10	40.74	3.30	161.05 40.15 2.16	15.23	22.08	4.73	67.28 27.23 4.33
5...	P_5	1.21	4.84	0.57	20.87 5.65 0.54	33.18	40.15	2.16	203.36 41.77 1.42	22.51	27.23	4.33	98.84 34.01 4.22
6...	P_6	1.13	5.65	0.54	27.06 6.42 0.51	36.97	41.77	1.42	246.55 42.37 0.78	30.11	34.01	4.22	137.07 40.57 4.04
7...	P_7	1.07	6.42	0.51	33.99 5.46 0.27	39.60	42.37	0.78	289.70 32.08 0.14	37.92	40.57	4.04	181.68 35.75 2 16
8...	P_8	0.78	5.46	0.27	39.72	41.13	32.08	0.14	321.92	45.84	35.75	2.16	219.59

Mr. Cain. ferences, and he is to be congratulated on having derived such a brief and valuable method of computation.

In conclusion, a few words may not be inappropriate concerning the advantage of the method of single loads in arch analysis. This consists not only in an increased accuracy, but is the only practicable method by which maximum fiber stresses can be ascertained. Thus, after the equilibrium polygons for the various unit loads have been drawn, the exact position of the live load to give the maximum stress at any section of the arch can be at once determined from the figure. The writer has shown* the great variations in stress at the critical sections of an arch as a live load moves across it, and has indicated probable positions for this load to cause maximum fiber stress at certain sections. Evidently, however, the positions will depend on the shape of the arch, and will thus vary as the arch is thicker or thinner and also with the curves of the intrados and extrados. The method of single loads gives certainty, the usual method uncertainty, in endeavoring to find maximum stresses. It is feared, therefore, that of the tens of thousands of concrete arches built in the United States, the maximum stresses in but very few have been ascertained. The subject is a vital one, and of practical importance.

* "Theory of Solid and Braced Elastic Arches."

MEMOIRS OF DECEASED MEMBERS

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

BENJAMIN MORGAN HARROD, Past-President, Am. Soc. C. E.*

DIED SEPTEMBER 7TH, 1912.

Benjamin Morgan Harrod was born in New Orleans, La., on February 19th, 1837. His father, Charles Harrod, came to that city in 1809, and made it his home. His mother, Mary Morgan, was a native of Louisiana, and the daughter of Benjamin Morgan who had come from Pennsylvania in the late years of the previous century and engaged in business and planting.

After attending the local schools, he prepared for college at Flushing, N. Y. He afterward entered Harvard University and was graduated in the class of 1856, with a well-formed intention of following a technical or constructive profession.

After a year or more of preparatory study, Mr. Harrod secured employment as Draftsman, in the United States Engineer Office in charge of the construction of forts and light-houses on the Gulf Coast, from the Mississippi River to the Rio Grande River, and subsequently was appointed Assistant Engineer. In 1859 he commenced practice in New Orleans, as an Engineer and Architect.

In April, 1861, he enlisted as a private in the Confederate Army, and was soon commissioned as Lieutenant in an Artillery Regiment, being detailed on engineer duty. He served as Brigade and Division Engineer with the command of Gen. M. L. Smith in the fortification and defense of New Orleans and Vicksburg. When exchanged after the surrender of the latter place, he was commissioned as Captain of Engineer troops in Virginia, was engaged in the defense of Petersburg and Richmond, and followed the fate of that Army to the surrender at Appomattox, in 1865. He then resumed professional practice in New Orleans.

In 1877 Major Harrod was appointed Chief State Engineer of Louisiana, with the late H. B. Richardson and T. S. Hardee, Members, Am. Soc. C. E., as associates, on a board, the principal function of which was the construction of a system of levees to protect the alluvial regions of the State from overflow.

In 1879 he was appointed an Engineer member of the Mississippi River Commission, charged by the United States Government with the survey of the Mississippi River and its tributaries, and the improvement of the main streams from the junction of the Ohio to the

* Memoir prepared by the following committee: Frank M. Kerr, Sidney F. Lewis, and Arsene Perrilliat, Members, Am. Soc. C. E.

head of the Passes, with special reference to confinement and its conservative influence on navigation. The importance of this latter part of the work of the Commission was subsequently recognized by the Government, and the repair and building of levees was made mandatory. It has been one of the most beneficent and remunerative works of the age, affecting the reclamation from overflow of 30 000 sq. miles of territory of unsurpassed fertility, and its development in population, agriculture, commerce, and wealth.

From 1888 to 1892 Major Harrod was City Engineer of New Orleans, and, subsequently, Advisory Engineer for the drainage, sewerage, and water-works systems of that city; and from 1897 to 1902, he had charge of the drainage, both designing and constructing.

In 1903 he was appointed, with Gen. C. W. Raymond and John Bogart, Members, Am. Soc. C. E., as the first United States Delegates to the International Congress of Navigation, held that year at Düsseldorf, Germany, but was prevented by business from attending its sessions.

In 1904 he was appointed a Member of the Panama Canal Commission, and served until 1907, when the type of the canal was determined and the charge of the work was transferred to the present Commission. His subsequent work was as a Consulting Engineer in New Orleans.

He married, in 1865, Miss Harriet Shattuck Uhlhorn, and, in 1883, Miss Eugenia Uhlhorn, both of New Orleans, the latter surviving him.

The degree of LL.D. was conferred on him by Tulane University, in 1906, as a

> "Graduate of Harvard University fifty years ago; President of the American Society of Civil Engineers; expert specialist and virile all-round man; friend of Tulane University and of all movements to better this City. In deepening a river and now in cutting an isthmus, his work has ever been to bring men closer in commerce, in friendship, and in mutual helpfulness."

Major Harrod was also a Member of the Association of Harvard Engineers, and its Vice-President in 1909, as well as a Member of the Louisiana Engineering Society.

He was a man of sterling qualities and strong individuality, but at all times generous and kind, and the friend of the worthy young aspirant for advancement in the Profession. Recognized, as he was, as a man of letters, of science, and of pronounced artistic taste, and as a high-toned, patriotic, public-spirited, loyal citizen, his loss will long be felt by the many who in life had the privilege of knowing him and in consequence admiring him.

Major Harrod was elected a Member of the American Society of Civil Engineers on April 4th, 1877. He served as Director from 1892 to 1894, as Vice-President in 1895-96, and as President in 1897.

THOMAS MOORE JACKSON, M. Am. Soc. C. E.*

Died February 3d, 1912.

Thomas Moore Jackson, son of James Madison and Caroline Moore Jackson, was born at Clarksburg, W. Va., on June 22d, 1852. He received his early education at the public schools and at the Northwest Academy, of Clarksburg, and at Bethany College. He afterward entered Washington and Lee University where he took a special course in civil and constructive engineering, and from which he was graduated in June, 1873, with high honors, with the degree of Civil Engineer.

From 1874 to 1875, Mr. Jackson served as Chief Engineer of the Middle Island Railroad, in West Virginia, and from 1875 to 1879, he was Chief Engineer of the Weston and West Fork Railroad. In 1879, he was appointed First Assistant Engineer of the Iron Valley and Morgantown Railroad, later becoming Chief Engineer.

Mr. Jackson resigned this position to accept that of Chief Engineer of the Tunnelton and Kingwood Railroad, in West Virginia, where he remained until 1881, when he was appointed Engineer in Charge of mines at Wilsonburg, Clarksburg, Flemington, and Gaston, W. Va. Mr. Jackson had been engaged in mining engineering in various parts of West Virginia and Pennsylvania since 1875, and in co-operation with I. C. White, State Geologist, he opened and developed many of the coal, oil, and gas territories in various sections of West Virginia, being one of the pioneers in this field.

In 1882, Mr. Jackson was appointed Chief Engineer of the West Virginia and Pennsylvania Railroad. He held this position until 1885, when he resigned. He was retained, however, by the Company as Consulting Engineer for many years.

In 1887, he was made Assistant Engineer of the Clarksburg Water-Works and was also Engineer in Charge of geological maps of West Virginia and Pennsylvania, working on the coal fields of these States. In 1888, he was engaged in building coke plants at Wilsonburg, Clements, and Clarksburg, W. Va., and, in 1889, he went to Morgantown, W. Va., as Chief Engineer of the water-works and natural gas plant at that place.

In 1889 the Chair of Civil and Mining Engineering was established at the West Virginia State University, and Mr. Jackson was placed at the head of that Department. He remained in this position until 1891, when, the School of Engineering having been firmly established, he resigned against the protests of the Board of Regents and the Faculty of the University, to take up the active practice of his Profession.

After leaving the University, Mr. Jackson served as Chief Engineer of several railroads, among which was the narrow-gauge road from

* Memoir prepared by the Secretary from papers on file at the Society House.

Clarksburg to Weston, afterward known as the West Virginia and Pittsburg Branch of the Baltimore and Ohio Railroad.

Being prominent in the development of the coal, oil, and gas industries of that section of West Virginia about Clarksburg, and realizing the importance of a more direct route to the Ohio River from the latter place for the transportation of these products, Mr. Jackson undertook and was directly responsible for the building of the West Virginia Short Line Railroad, from Clarksburg to New Martinsville, which was completed and opened to traffic in 1901. He was President of this road until its purchase by the Baltimore and Ohio Railroad.

In 1910, Mr. Jackson organized and was made President of a company to build the Clarksburg and Northern Railroad to extend from New Martinsville, by way of Middlebourne and Salem, to Clarksburg, W. Va. Construction work on this road was begun in 1912, and the grading completed between New Martinsville and Middlebourne. Mr. Jackson had just returned to Clarksburg from New Martinsville, where he had been on business in connection with the new road, when he was seized with a severe attack of heart trouble which caused his death on February 3d, 1912, after a short illness.

In 1884, he was married to Miss Emma Lewis, daughter of Judge and Mrs. C. S. Lewis, who, with one daughter, survives him.

Mr. Jackson was a courteous and cultured gentleman, friendly and companionable on all occasions, who was esteemed and admired by all who knew him. He was one of the most prominent and progressive citizens of Clarksburg and was well known among business and professional men throughout the State. Practically his entire life was spent in his home town, and he did much toward its development as an industrial center. His ability as a civil engineer and his great knowledge of geology enabled him to realize the importance of the vast gas, oil, and coal areas in and around his home section and also the necessity of developing them for the establishment of manufacturing industries. It was through his efforts that the Jackson Iron and Tinplate Mills, now the Phillips Sheet and Iron Mills, which is one of the most valuable industrial plants in West Virginia, was established in Clarksburg.

He was President of the Traders National Bank of Clarksburg until its merger with the People's Banking and Trust Company into the Union National Bank, and was one of the chief promoters of the Traders Hotel, the building of which gave the city an up-to-date hotel.

The degrees of Mining Engineer, Doctor of Science, and Civil Engineer had been conferred on him by Washington and Lee University and by the West Virginia State University, respectively, and, as a member of Governor Fleming's staff, he had received the honorary title of Colonel.

Mr. Jackson was elected a Member of the American Society of Civil Engineers on June 9th, 1891.

WILLIAM FREDERICK LOCKWOOD, M. Am. Soc. C. E.*

Died August 22d, 1912.

William Frederick Lockwood was born in New York City on July 4th, 1867, and, after attending the public schools, learned the trade of steam fitter. He entered the Engineering Department of what is now the Suburban Branch of the Third Avenue Elevated Railway during its construction in August, 1888, and on its absorption by the Manhattan Railway Company, in 1891, was taken into the Chief Engineer's Office, serving successively as Draftsman and Assistant Engineer until September 16th, 1900, when he was appointed Road Engineer, the duties of which office were the maintenance of the elevated railway structures.

While working as a Draftsman, Mr. Lockwood realized the limitations of his early education, and attended Cooper Institute in his spare time, receiving the degree of Bachelor of Science in 1894.

He was appointed Engineer of Maintenance of Way of the Interborough Rapid Transit Company on October 1st, 1905. His continuous term of service on the elevated roads covered twenty-four years, or practically his entire working life. During this time the roads were being extended, and improvements were being adopted, thus making the conditions of maintenance very burdensome, on account of the deeper foundations of buildings and numerous sub-surface structures, such as subways, conduit lines, high-pressure water mains, etc., which required the reconstruction of numerous foundations for the railway and the constant shoring of the structure at various points.

The adoption of electric motive power, requiring the reinforcement of the structure, was in itself a great task, and the constantly increasing street traffic rendered all such work more and more difficult. It is easily seen that such conditions would tend to keep a man of Mr. Lockwood's conscientious temperament very busy.

During all this time he had the care of his mother's family as well as his own. Time for study and almost for recreation was practically denied him.

His personality was somewhat unique, in that it was so fine. Of clear intellect, inflexible integrity, tireless industry, great patience, and a charming presence, he was a power for influence and accomplishment.

Mr. Lockwood died after a short illness following an operation for appendicitis. He is survived by his widow and three children, Ruth, Blanche, and Constance, and four sisters.

Mr. Lockwood was elected a Member of the American Society of Civil Engineers on October 4th, 1910.

* Memoir prepared by George H. Pegram, M. Am. Soc. C. E.

EDWARD MOHUN, M. Am. Soc. C. E.*

DIED OCTOBER 23D, 1912.

Edward Mohun, was born at Chigwell, England, on September 3d, 1838. In 1863, he came to Victoria, B. C., Canada, and during 1863 and 1864 was engaged as Assistant to the Surveyor General.

From 1864 to 1871, he was employed mainly on official surveys: laying out roads, etc., extending from Comox to Sooke, Nicola, South Thompson, and Bonaparte Valleys; surveying the Lower Fraser from Chilliwack to its mouth, Burrard Inlet, Squamish River, etc.; exploring Queen Charlotte Islands south; and making surveys for a waterworks system for Victoria and for the Wellington and Departure Bay Railway.

In 1871 and 1872, Mr. Mohun was engaged as Divisional Engineer on the Eagle Pass and Yellowhead Pass surveys for the Canadian Pacific Railroad. He also drove the first stake and made the first survey for this road on the Pacific Coast. In 1873, he explored the north end of Vancouver Island, and in 1875-76 was engaged in Government Surveys on that island and on the Lower Fraser River.

In 1876, he received the appointment of Surveyor to the Dominion and Provincial Joint Indian Commission, serving as Chief of the Survey from 1877 to 1884.

In 1885 Mr. Mohun was engaged on the reclamation and diking of 7 000 acres on the Lower Fraser River. While on this work he invented the sluice-box which was adopted by the Government and has been in use for more than 15 years, effecting the saving of many thousands of dollars.

In 1886, he made a test of the woods of British Columbia for which he received his proudest possession, a certificate signed "Albert Edward" and a medal. These tests agreed with similar tests made by the United States Government some twenty years later, and are the basis on which all strains in bridges built by the British Columbia Government are supposed to be calculated.

In 1887, Mr. Mohun designed and constructed the first portion of the sewerage system of Vancouver, B. C., the second portion being built under his supervision in 1889 and 1890.

In a competition for the design of a sewerage system for Victoria, B. C., in 1890, he was awarded the premium over eight competitors from Eastern Canada, the United States, and England, and was appointed Chief Engineer on its construction. As a result of the installation of this system, the death rate of Victoria was reduced 25% in three years.

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* Memoir prepared by the Secretary from information on file at the Society House.

From 1893 to 1895, Mr. Mohun was employed as Provincial Government Railway Inspector, and, in 1895, he was appointed Engineer in Charge of the Pitt Meadows Dikes.

In 1898, he made a report to the Government on the sanitary condition of Rossland, Trail, Nelson, Kaslo, Revelstoke, Kamloops, New Westminster, Cumberland, Wellington, and Nanaimo. In the same year he entered the office of the Engineer of Public Works, and, in 1908, was gazetted as First Assistant Engineer. During this time he was engaged in the design of various public works, and since 1896 he had also acted as Consulting Engineer to the Provincial Board of Health, to which, in 1892, he had reported on the "Bacterial Treatment of Sewage." His conclusions on this subject were practically confirmed a few years later by the British Royal Commission on Sewage Disposal.

Mr. Mohun was the author of many engineering papers and of tables which are in constant use on the Pacific Coast. He was a Member of the Canadian Society of Civil Engineers, a Gzowski Medallist, and was also at one time Member of the Council of the Society. He was also a Member of the Royal Sanitary Institute, and for fifteen years was a Justice of the Peace for the Province.

Mr. Mohun was elected a Member of the American Society of Civil Engineers on April 6th, 1892.

WILLIAM MADISON MYERS, Assoc. M. Am. Soc. C. E.*

Died April 4th, 1912.

William Madison Myers, son of William H. and Mary Jane (Harman) Myers, was born near Winchester, Va., on April 4th, 1873. His parents, who lived on a fine farm in the Shenandoah Valley, perceiving that the son had a fondness for books and study, and realizing the value of education, determined to fit him for a professional life. After availing himself of all the advantages of the country schools, Mr. Myers entered the Winchester High School, from which he was graduated in 1891. In the fall of the same year he entered Washington and Lee University, and was graduated from that institution in 1895, with the degree of Civil Engineer.

For some time after his graduation, he taught in the Graded Schools of Frederick County, Virginia, being engaged occasionally on local surveying and engineering work. In 1897 and 1898, he was in charge of road improvements between Leesburg and Harper's Ferry, Va., under Frank Conrad, Civil Engineer.

* Memoir prepared by Mrs. Bessie B. McCann.

In 1898, Mr. Myers accepted an offer from the late Lewis Kingman, M. Am. Soc. C. E., then Chief Engineer of the Mexican Central Railway, to go to Mexico as Transitman on the location of the Tampico Division of that railway, being also at times in charge of a party locating reservoir sites, etc. In 1900 he was appointed Assistant, under Mr. W. C. Harris, on the construction of the Parral Extension of the same road, and, in 1901 and 1902, he was with Mr. O. G. Bunsen, as Transitman and Topographical Draftsman, on the location of the Colima Extension.

From 1902 to 1904, Mr. Myers was employed as Division Engineer on the construction of the San Pedro Extension of the Mexican Central Railway, and, while in this position, had charge of the construction of 50 km. of the road, as well as of all the Division yards and buildings. He remained in this position until the work was completed, when he returned to Virginia to visit his old home. After seven years' service on the Mexican Central Railway, under Mr. Kingman, the latter wrote of him:

"Mr. Myers is a gentleman and an educated engineer, and has done all his work, while under my jurisdiction, with credit to himself and to the entire satisfaction of the Railway Company. I take pleasure in recommending Mr. Myers. He is honest, truthful, and considerate. He is industrious, temperate, and careful in his work. He understands the classification of material, and has always done the right thing with the contractors and the Railway Company."

In 1904, Mr. Myers was appointed Transitman and Topographical Draftsman on the Florida East Coast Railway, but in less than a year, because of the climatic conditions, was compelled to resign this position and return to his home to recuperate.

In 1905, he went back to Mexico, as Division Engineer, under Mr. Kingman, on the construction of the Colima Extension of the Mexican Central Railway from Tuxpan to Colima. This work was very heavy and included the construction of three tunnels in a distance of 7 km. In 1906, Mr. Myers accepted a position with Pierson and Sons, of the Puebla Tramway, Light and Power Company, at Puebla, Mexico, his work consisting of city surveys, and surveys and estimates for water-works, hydro-electric plants, electric lines, etc. Of him and his work at this place, Mr. Pierson has written:

"I consider him an able and experienced railway and general engineer, competent to take charge of important work. * * * As to Mr. Myers' character, I have the highest opinion of him. He is of good family, sober, reliable, and conscious of the dignity of his Profession."

After spending three years in Mexico, Mr. Myers returned to his home in Virginia and was again engaged in local engineering around Winchester. In 1910, he entered the employ of the Virginia State

Highway Commission, under Mr. P. St. J. Wilson, having been located at Buena Vista, Big Stone Gap, and Fairfax, Va. While engaged at the latter place, in January, 1912, he was taken ill with typhoid fever and removed to the Georgetown University Hospital, where he remained under treatment for two months. Failing to improve, he was removed to his home in Winchester, where he died on April 4th, 1912.

Mr. Myers was always a student, and surrounded himself with the latest works on all branches of engineering. He was painstaking, and all his work and papers show that great care and precision were his marked characteristics. He was a member of the Presbyterian Church, and carried his love of God and man in his work, always living his religion.

Mr. Myers was elected an Associate Member of the American Society of Civil Engineers on October 3d, 1906.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

CHARACTERISTICS OF CUP AND SCREW CURRENT METERS PERFORMANCE OF THESE METERS IN TAIL-RACES AND LARGE MOUNTAIN STREAMS STATISTICAL SYNTHESIS OF DISCHARGE CURVES.

By B. F. Groat, Assoc. M. Am. Soc. C. E.
To be Presented February 5th, 1913.

During the summer and autumn of 1912 the writer ran a number of efficiency tests on two of the 6 000-h.p. hydraulic turbine units recently installed in the power-house of the St. Lawrence River Power Company, at Massena, N. Y. On one of the units, two sets of tests were run, one prior, and one subsequent, to cutting off several feet of the draft-tubes. On the other unit, one set of tests was run after the draft-tubes had been shortened, as had been done in the case of the other turbine unit.

In all, some 40 000 instrumental readings, including gauge readings, were made. Of these, 7 000 were complete velocity observations by current meters, while 4 000 additional readings on a Pitot tube furnished about 100 complete velocity observations for comparison with simultaneous readings by the current meters.

Owing to the nature of the tail-races, which discharge into the Grasse River, a weir test was out of the question. Before beginning

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

the tests, therefore, it became necessary to decide between the current meter and Pitot tube as instruments for measuring the flow.

In favor of the current meter is its simplicity and the facility with which it may be used in tail-races. Against it is the fact that, notwithstanding the large number of observers who have experimented with it and the large number of records which have been made by it, there is still a great deal of well-founded skeptical discussion as to the accuracy of the instrument when the records are based solely on the still-water rating.

The Pitot tube has been not only a subject of adverse criticism but also an instrument about which much positive ignorance seems to exist. It would seem, however, that the recent demonstrations of the accuracy of Darcy and Bazin's experiments, 1865, by Messrs. William Monroe White,* L. F. Moody, Gardner S. Williams, John R. Freeman, and others, should sufficiently dispel all doubts as to the reliability of the instrument. On the other hand, where the velocity of the water is variable from moment to moment, a large number of readings must be taken and reduced to a mean square root in order to obtain sufficiently close approximations to the average in any one velocity determination.

After a final summary of advantages and disadvantages, it was decided to use both the screw and cup types of current meter, and to check their records, if necessary, with a Pitot tube.

E. E. Haskell, M. Am. Soc. C. E., Director of the College of Civil Engineering, Cornell University, made up one of his screw type meters especially for the series of tests to be undertaken, and a new Gurley-Price meter of the cup type (No. 600) was purchased from the makers at Troy. Master Mechanic D. J. Jones, of the Power Company, made up a Pitot tube, following as nearly as possible the description of Tube *N* given in Mr. White's paper, mentioned above.

The following are the general conclusions concerning current meters which have been drawn from the tests with the foregoing equipment:

1.—†When a cup meter is run in perturbed water it will register

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* "The Pitot Tube; Its Formula," by William Monroe White, *Journal*. Association of Engineering Societies, Vol. XXVII, 1901, p. 35.

† In a discussion of a paper on current meter and weir discharge comparisons by Edward C. Murphy, Assoc. M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. XLVII, 1902, p. 370, Charles H. Miller, M. Am. Soc. C. E., describes certain experiments on screw and cup meters in the following language :

"The Haskell meter was lowered from one side of the skiff and one of the Price meters

a larger number of revolutions per second than a perfect still-water rating would indicate.

2.—†When a screw meter is run in perturbed water it will register a smaller number of revolutions per second than a perfect still-water rating would indicate.

3.—†In the foregoing sense, a cup meter is affected relatively to a much greater extent than a screw meter. In the tail-races at Massena, as an average, the cup meter was affected to the extent of 6%, while the Haskell meter was affected mostly by less than 1 per cent. In boilers of considerable violence the cup meter may easily over-register by 25%, while the screw meter will under-register by not more than 3 or 4 per cent.

4.—Either type of meter when run in perturbed water will give uniform records in equal times provided these times are sufficiently long, the flow of the water itself being subject to an established regimen.

5.—If both types of meter are used simultaneously in perturbed water, the disparity between the discrepant velocities thus determined by the still-water rating may be taken as a basis for correcting the discrepant velocities.

6.—The average corrections thus obtained for the Haskell meter when run with the Gurley-Price meter in the tail-races at Massena varied from 0.5 to 0.9 of 1%, while the corresponding corrections for the Gurley-Price meter were about six times larger. Comparisons with the Pitot tube furnished substantially the same corrections as those obtained by comparison of the meters.

7.—It would seem to follow that current-meter observations based on still-water ratings without further correction should be made with great caution. On the other hand, it seems certain that the correction

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from the other side, both to the same depth (about 5 ft.). The registrations or revolutions of each, for periods of from 5 to 10 minutes, were noted; then, for similar periods of time, the skiff was caused to rock from one side to the other at short intervals, and the registrations recorded, the movement of the skiff giving to the meters a vertical motion of from 1 to 2 ft. Under this motion the revolutions of the Price meter increased, while those of the Haskell meter decreased, but in a less proportion than the increase of the Price meter, showing that the error due to instability of support is greater in the Price than in the Haskell meter. Lack of time prevented more extended observations, and in different velocities of current, which probably would have evolved some fixed difference due to the different construction of the meters in question."

Conclusions 1, 2, and 3, of the present paper seem to have been fairly stated by Mr. Miller, but as applied only to instability of support. They are here advanced as inherent characteristics of these different types of meter, and extended to apply, not only to cases of instability of support, but also to these current meters generally, however supported, in flowing water.

for a cup meter when run at a good meter station on an open river is not large, while the corresponding correction for a screw meter may be negligible.

Plate CXXXIV shows the results of a number of ratings of the Haskell meter. The general method was similar to that adopted by the United States Lake Survey, as described in the reports by F. C. Shenehon, M. Am. Soc. C. E.,* covering his current-meter observations. The meter was suspended by either cable or rod from a bowsprit at the head of a light skiff. The skiff, with an observer, was then towed back and forth over a 200-ft. base at a given velocity, one such double run constituting a velocity observation. The total time and total number of revolutions then stand for the transit of a 400-ft. base in perfectly still water.

The first four ratings, on June 29th and 30th, 1911, were made along the longitudinal center line of Lock No. 15 in the old Cornwall Canal, at Cornwall, Ont. Both lock gates were open, but the north miter leaf of the lower gate, opening into the St. Lawrence River, was not fully into its recess in the north wall. The meter was about 4 ft. below the surface of the water and the depth in the lock was 9 or 10 ft. The water in the lock had a tendency to surge at times. Sometimes there was considerable velocity through the lock, and at other times very little.

The ratings of August 12th and October 3d, 1911, were made in a bay of the Grasse River, opposite the power-house of the St. Lawrence River Power Company, at Massena, N. Y. The meter was about 2.25 ft. below the surface of the water, and the average depth under the base line was about 7 ft., with a shallow spot near the east end about 5 ft. deep. There was very little current, but at times there were surges due to the operation of the power-house just across the river.

The ratings of July, 1912, were made under the direction of Professor Arthur M. Greene, Jr., at the rating station of Rensselaer Polytechnic Institute, Troy, N. Y., after the meter had been used extensively at Massena and in North Carolina and Tennessee. The meter had been overhauled twice at Cornell University before these ratings, but long after those at Massena. The meter was also provided with a new shaft. After running a complete set of tests at Troy, the new shaft was put in and another set of tests was made.

* As described in Reports, Chief of Engineers, U. S. Army, 1900 to 1904, inclusive.

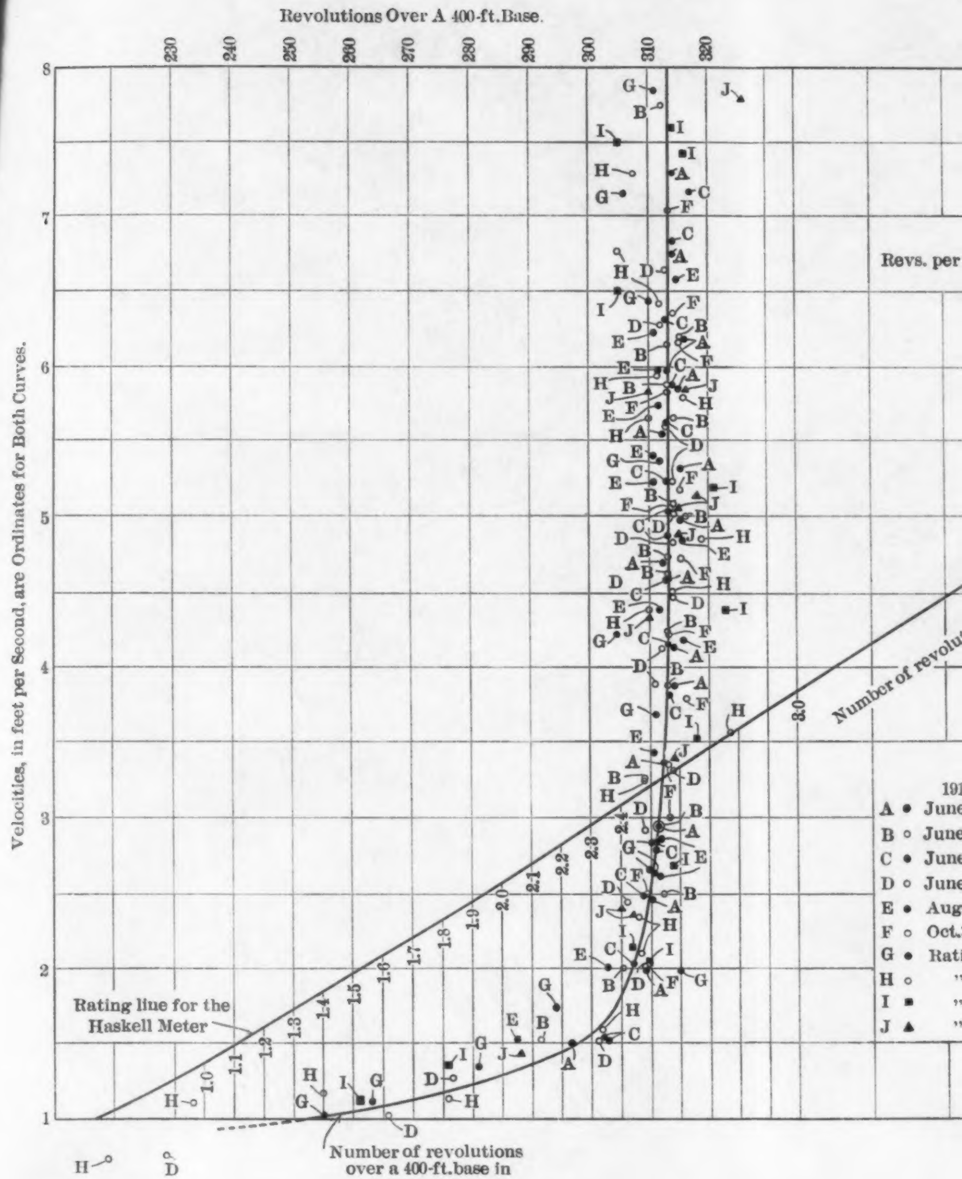
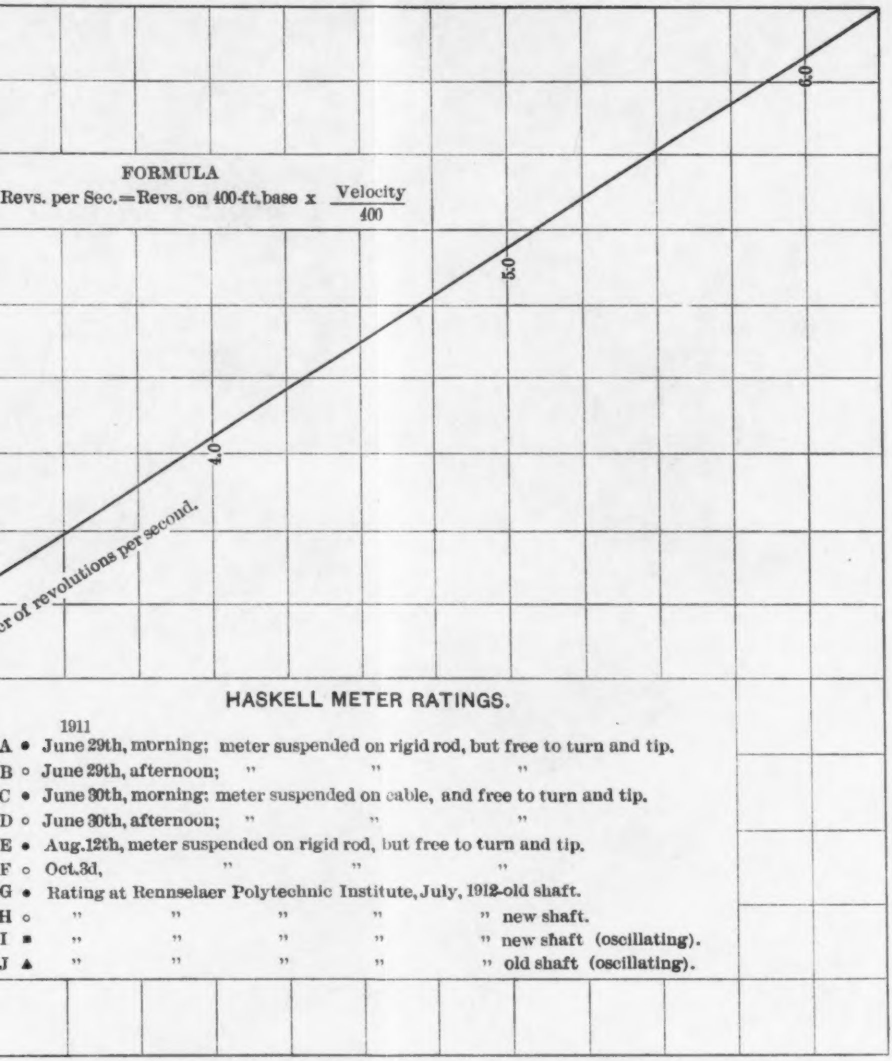

Revolutions Over A 400-ft. Base.
Velocities, in feet per Second, are Ordinates for Both Curves.
Revs. per
Number of revolu
Rating line for the
Haskell Meter
Number of revolutions
over a 400-ft. base in
perfectly still water.
A • June
B ○ June
C • June
D ○ June
E • Aug
F ○ Oct.
G • Rati
H ○ "
I ■ "
J ▲ "

PLATE CXXXIV.
PAPERS, AM. SOC. C. E.
DECEMBER, 1912.
GROAT ON
CUP AND SCREW CURRENT METERS.



The curve of revolutions over a 400-ft. base was plotted from the first four ratings in the Cornwall Canal. It will be seen from Plate CXXXIV, however, that the subsequent ratings do not materially alter the position of this line, up to velocities of 5 or 6 ft. per sec. If the correction were made, the number of revolutions, as shown by the line, would be increased only ¼% at 4 ft. per sec., and reduced only ½% at about 8 ft. per sec. In fact, it may be seen that, out of about 100 double runs in the canal lock and Grasse River, not one plotted point differs by more than 1% from the line of revolutions as determined from the first four ratings for velocities of more than 1½ ft. per sec. For velocities of less than 1½ ft. per sec. the line of revolutions is so steep that errors of somewhat larger magnitude are to be expected. In the oscillating tests, the meter was oscillated parallel to the motion of the car, and covered an amplitude of 5¾ in. for every 8-ft. advance of the car. Transverse oscillations reduce the number of revolutions over a given base. The relations between velocity and revolutions over a 400-ft. base were then plotted, as shown on Plate CXXXIV, and from the resulting curve the rating line for still water was located and drawn, also as shown on that plate.

After having drawn the rate line, a reduction diagram, Plate CXXXV, may be prepared, from which velocities may be read directly when referred to time and revolutions. This obviates either the necessity for reducing revolutions to revolutions per second, or of taking a fixed time for observing velocities, which is inexpedient where velocities are variant.

Plate CXXXVI shows the Gurley-Price meter rating curves, obtained in a manner precisely similar to that used for the Haskell meter. In fact, the ratings were frequently made on the same day as for the Haskell meter.

In order that there may be no misunderstanding concerning these ratings, it may be stated that the tests were severe, when considered as a whole. Some of the ratings were made at a depth of 4 ft., some at a depth of 2 ft. 3 in., some in windy weather, some in calm weather, and several different observers acted at different times.

Under all these diverse conditions it may be seen that the Haskell meter, in the Massena and Cornwall ratings, never varied by more than 1% in any individual observation, and that rating lines drawn for different ratings would differ by only 0.2 or 0.3 of 1% at the greatest.

The Gurley-Price meter exhibits a much greater range of variations, 5 or 6% up to velocities of 5 ft. per sec., which were considerably in excess of the mean velocities in the tail-races where the meter was used.

It is not contended here that a satisfactory still-water rating under perfect conditions cannot be obtained with a meter of this type; it is intended, however, to show that varying conditions of water produce relatively larger variations in the records of the cup meter than in those of the screw meter.

On comparing the maker's rating curve with the several ratings in the canal and in the bay of Grasse River, it will be seen that the number of revolutions of the Gurley-Price meter over a 400-ft. base is generally higher in the canal and bay than in the maker's rating. Hence it is fair to suspect that a perfect still-water rating line of a cup meter is a line of minimum number of revolutions for such a meter, and that, when the water is disturbed in any degree, the number of revolutions over a fixed base will always be increased, other things being equal.*

Similarly, the contrary suspicion may be entertained, that a perfect still-water rating line of a screw meter is a line of maximum number of revolutions for such a meter, and that, when the water is disturbed in any degree, the number of revolutions over a fixed base will always be decreased, other things being equal.

Since, under similar circumstances, the deviations of the Gurley-Price meter are about six times as large as those of the Haskell meter, but in the contrary sense, it would seem fair to conclude that when the meters are run simultaneously in flowing water, one-seventh of any difference in the velocities thus determined from the still-water ratings should be attributed to a deviation of the Haskell meter, while six-sevenths should be attributed to a simultaneous deviation of the Gurley-Price meter.

Thus, if the difference between the velocities determined by the Haskell and Gurley-Price meters should be 0.42 ft. per sec., 0.06 ft. per sec. should be added to the record of the Haskell meter, while 0.36 ft. per sec. should be deducted from the corresponding record of the Gurley-Price meter. In strictness, this correction should be

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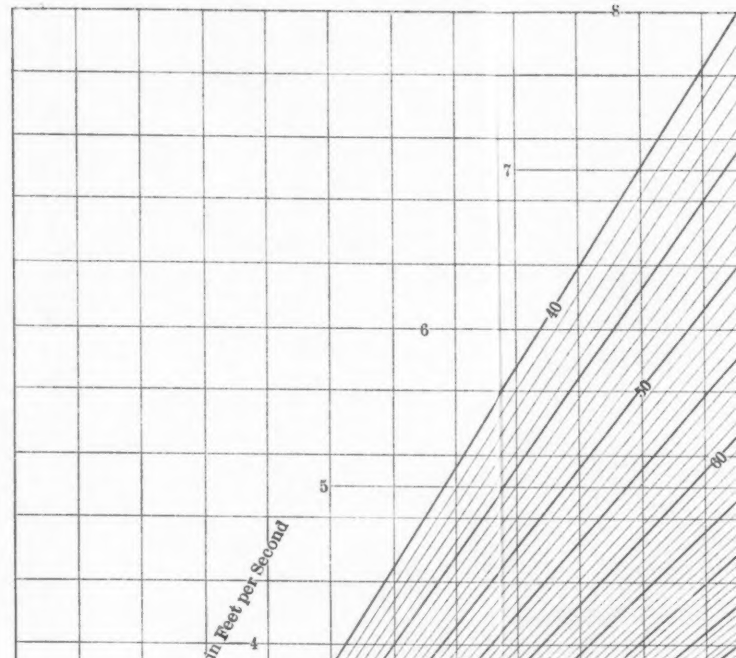
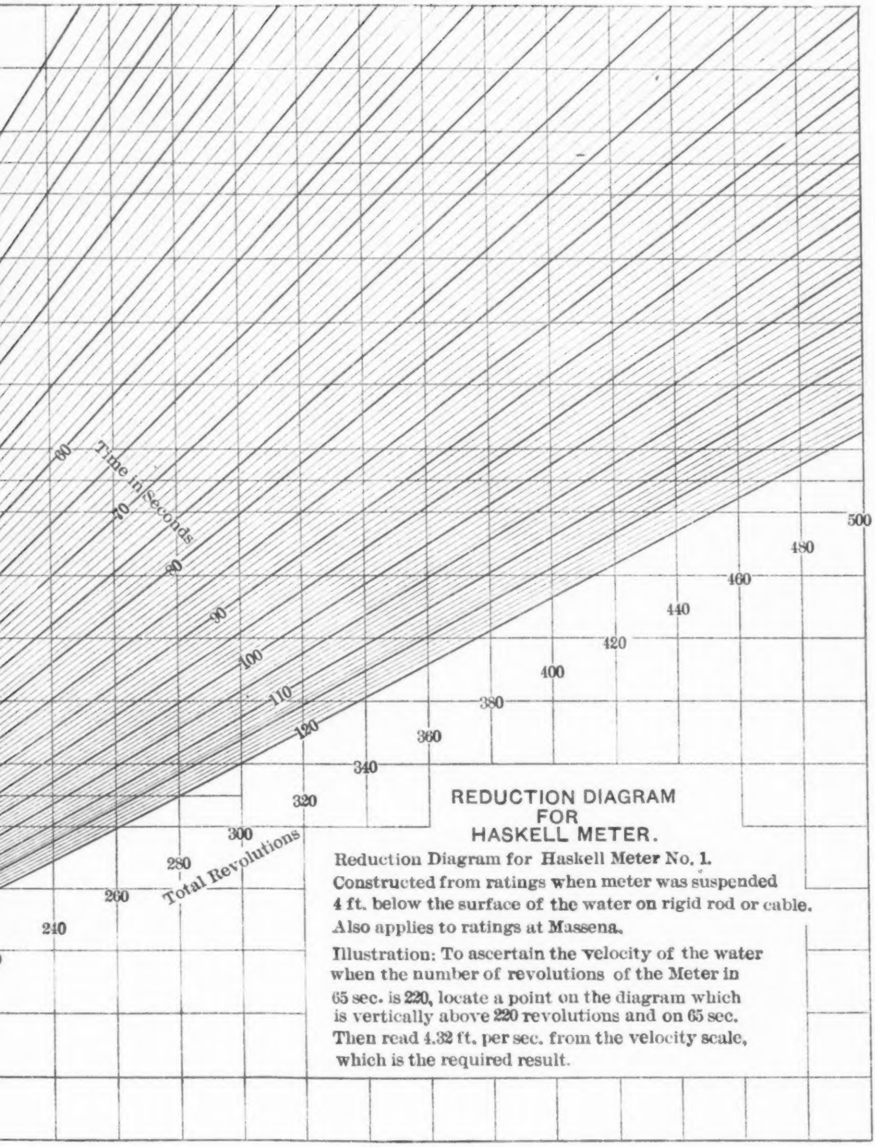
* Certain interesting experiments by Professor L. F. Moody, in the rating station at Rensselaer Polytechnic Institute, seem to agree with the writer's experiments on this point and confirm the general conclusion.



Velocity, in Feet per Second
Time, in Seconds
1
2
3
4
5
6
7
8
40
50
60
20
40
60
80
100
120
140
160
180
200
220
240
40 45 50 55 60 65 70 75 80 85 90 95 100 105 110 115 120

PLATE CXXXV.
PAPERS, AM. SOC. C. E.
DECEMBER, 1912.
GROAT ON
CUP AND SCREW CURRENT METERS.



REDUCTION DIAGRAM
FOR
HASKELL METER.

Reduction Diagram for Haskell Meter No. 1.
Constructed from ratings when meter was suspended 4 ft. below the surface of the water on rigid rod or cable.
Also applies to ratings at Massena.

Illustration: To ascertain the velocity of the water when the number of revolutions of the Meter in 65 sec. is 220, locate a point on the diagram which is vertically above 220 revolutions and on 65 sec. Then read 4.32 ft. per sec. from the velocity scale, which is the required result.

applied to revolutions per second, but, as the number of revolutions is so nearly proportional to the velocity, and as the corrections are relatively small, no serious error results. The particular ratio here determined, of course, applies only to the two individual meters as used in the tail-races at Massena by the writer.

In order to establish more satisfactorily the principle mentioned above, the boat was rocked during several of the runs of a rating, imparting to the meter which was being rated an oscillation in an arc about 3 ft. long and about $2\frac{1}{2}$ ft. in radius, and at right angles to the direction in which the meter was being drawn through the water. The result was that the Gurley meter was found to increase its number of revolutions by 15% while the Haskell meter reduced its number of revolutions by $2\frac{1}{2}$%, practically the same ratio as that deduced above from the rating curves. The effect of longitudinal oscillations was tested also. The result of special tests of this character at Rensselaer Polytechnic Institute, may be seen on Plate CXXXIV. Subsequently, other tests on a new Haskell meter showed that longitudinal oscillations of 22 in. in 6 ft. advance of car retarded the meter slightly, up to velocities of about 2 ft. per sec., above which the meter is accelerated, the error being less than 2% at 9 ft. per sec. The net result in perturbed water, however, seems to be slight retardation in all cases.

It is now desirable to show how these principles have been used to correct the current meter records actually observed in the tail-races during the turbine tests referred to at the outset. In order to do this properly, it will be necessary to state another conclusion, based on the current meter records, which the writer hopes to take up more fully in the future under another title. The conclusion is as follows:

Under an established regimen, the distribution of the flow of water in a tail-race obstructed by stilling racks is fixed in character, the ratio of the average velocity at any given point to that at any other given point in the race being constant. This proposition applies over a considerable range in the total amount of discharge, but supposes the actual wetted cross-section of the race to remain the same.

The foregoing proposition was proven from a number of the turbine tests by computing and compiling the relative velocities at the 84 meter points in the races where velocity observations were taken. A turbine unit discharged through two tail-races, each about 15 ft.

wide, while the depth of the water varied from 15 to 16 ft. The meter section of each race was divided into six equal vertical, and seven equal horizontal, strips, and the meter points were the forty-two intersections of the median lines of the strips. The verticals were numbered 1 to 6 from west to east, while the depths were numbered 1 to 7 downward.

To demonstrate the truth of the principle of a fixed distribution of flow in tail-races, and also as a verification of Conclusion No. 4, Tables 1 to 4 are given:

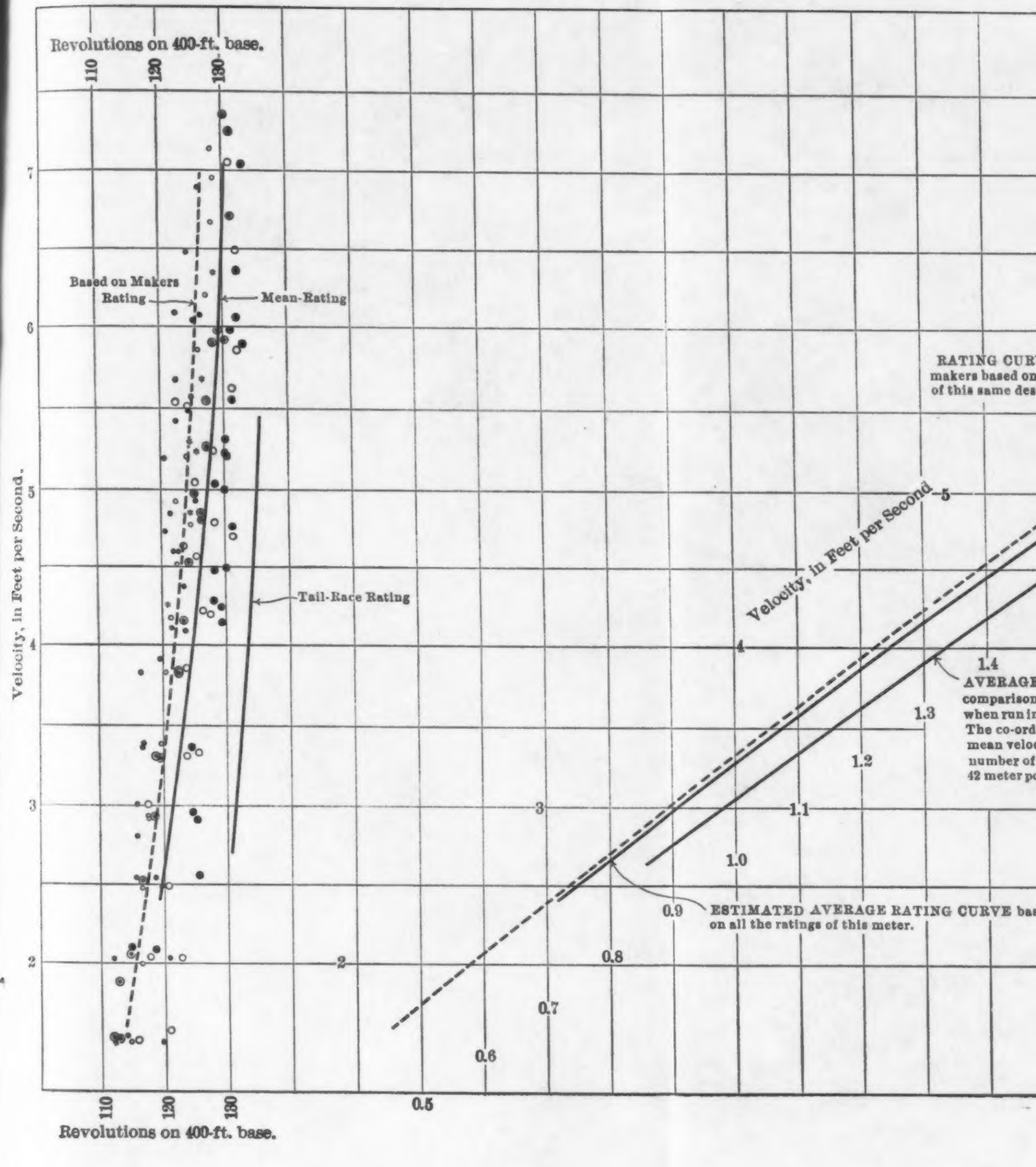
TABLE 1.—Gurley-Price Meter in West Race, Showing Constancy of Velocities Taken in Quick Succession. Velocities Were Observed with a Single Stop-Watch Provided with Two Second Hands.

Test of September 9th, at 0.82 gate.
The numbers are of revolutions per second.

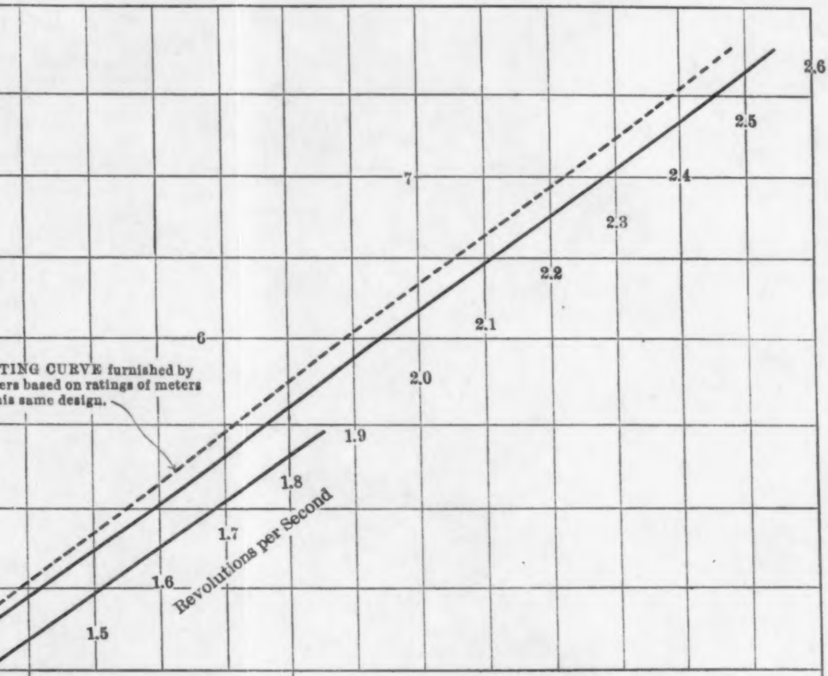
Depths.	Verticals.						Totals of lower set.
	1	2	3	4	5	6	
1	1.32	1.40	1.12	0.88	0.90	0.94	6.62
	1.33	1.42	1.16	0.89	0.87	0.92	
2	1.23	1.44	1.40	1.23	1.20	1.16	7.70
	1.24	1.44	1.40	1.24	1.22	1.16	
3	1.35	1.44	1.62	1.42	1.34	1.31	8.45
	1.35	1.44	1.59	1.41	1.35	1.31	
4	1.37	1.44	1.43	1.30	1.31	1.37	8.21
	1.36	1.44	1.43	1.30	1.32	1.36	
5	1.53	1.47	1.52	1.40	1.26	1.38	8.57
	1.49	1.48	1.53	1.42	1.27	1.38	
6	1.70	1.72	1.80	1.72	1.51	1.66	10.09
	1.69	1.72	1.80	1.72	1.50	1.66	
7	1.60	1.81	1.84	1.78	1.78	1.86	10.64
	1.58	1.80	1.84	1.78	1.79	1.85	
Totals of lower set.	10.07	10.74	10.75	9.76	9.32	9.64	60.28

The register was started first and the observer noted the pointer as it clicked from revolution to revolution of the meter. By synchronizing with the rythm of the register, it was possible for the observer to start the stop-watch almost exactly at the moment the register clicked to zero. Similarly, the primary second hand could be stopped after any desired number of revolutions while the secondary hand could be stopped at, say, twice that number. Thus two or more velocities in immediate succession could be determined for each meter point in the tail-races.

In support of the principle of fixed distribution, it may be stated that the percentages corresponding to the figures of Tables 1 and 2 at the 84 meter points agree remarkably well with the corresponding ones of Tables 3 and 4 after applying certain necessary small corrections. Tables 1 and 2 each represent an individual set of double discharge measurements in one of the races, while Tables 3 and 4 have



Revolutions on 400-ft. base.
110
120
130
Velocity, in Feet per Second.
Based on Makers Rating
Mean-Rating
Tail-Race Rating
RATING CUR
makers based on
of this same des
Velocity, in Feet per Second
AVERAGE
comparison
when run in
The co-ord
mean veloc
number of
42 meter p
ESTIMATED AVERAGE RATING CURVE bas
on all the ratings of this meter.



AVERAGE RATING CURVE based on comparison of Haskell and Gurley meters when run in tail-races during turbine tests. The co-ordinates refer practically to the mean velocity in the race and the mean number of revolutions per second at the 42 meter points.

RATING OF CURRENT METER NO. 299.

CORNWALL RATINGS, Lock 15, Old Cornwall Canal. Meter suspended about 6 ft. forward of stem of boat and 4 ft. below water surface. Water in lock 10 to 11 ft. deep. Base about 200 ft. long.

- • June 21'22. Meter suspended on cable, and free to turn and dip.
- ● June 23 P.M. Meter suspended on rigid rod, but free to turn and dip.
- ○ June 28 P.M. Meter suspended on cable, and free to turn and dip.

MASSENA RATINGS, made in bay of Grasse River opposite Power-House, over a 200-ft. base same as at Cornwall. The average depth was about 6 or 7 ft. when the elevation of the water surface is about 163 ft. above Sea Level, with a minimum spot of 4.5 or 5 ft. Meter suspended on rod about 6 ft. forward of stem of boat and about 2.25 ft below surface of water.

- ● August 12, P. M.
- ○ August 15, A.M.
- • September 29.

CURVE based

TABLE 2.—Haskell Meter in East Race, Showing Constancy of Velocities Taken in Quick Succession. The Comparisons are Not as Good as with the Gurley-Price Meter, Because the Velocities were Taken with Two Different Watches, One of which was Not very Reliable.

Test of September 9th, at 0.82 gate.
The numbers are of feet per second.

Depths.	Verticals.						Totals of lower set.
	1	2	3	4	5	6	
1	3.23 3.12	2.74 2.75	2.48 2.50	2.41 2.39	2.94 2.97	3.75 3.72	17.45
2	3.62 3.67	.... 3.49	.... 3.26	.... 3.27	.... 3.79	.... 4.01	21.49
3	4.12 4.22	3.96 3.98	3.53 3.79	3.91 3.98	4.13 4.17	4.09 4.10	24.24
4	4.21 4.23	3.92 3.95	3.64 3.64	3.73 3.70	3.88 3.91	4.07 4.05	23.48
5	4.30 4.35	3.90 3.94	3.73 3.73	3.68 3.63	3.61 3.64	4.25 4.28	23.57
6	4.77 4.80	4.25 4.31	4.74 4.75	4.61 4.68	4.18 4.22	4.44 4.49	27.25
7	5.07 5.12	5.27 5.27	5.32 5.35	5.05 5.01	4.98 4.99	4.61 4.58	30.32
Totals of lower set.	29.51	27.69	27.02	26.66	27.69	29.23	167.80

The two velocities at each meter point were taken in immediate succession in a manner similar to that described in the foot note under Table 1, except that two stop-watches were used. The observations were about 1 min. in duration for the first velocity and twice as long for the second velocity.

TABLE 3.—Relative Distribution of Flow in West Race of Unit No. 3 During Tests of July and August. Water Surface at Elevation 162.85.

Depths.	Verticals.						Totals.
	1	2	3	4	5	6	
1	2.16	2.18	1.84	1.53	1.50	1.97	11.18
2	2.11	2.41	2.32	2.07	2.02	2.22	13.15
3	2.04	2.37	2.46	2.31	2.18	2.14	13.50
4	2.13	2.29	2.34	2.20	2.26	2.36	13.58
5	2.19	2.41	2.57	2.29	2.27	2.67	14.40
6	2.65	2.87	3.00	2.85	2.42	2.68	16.47
7	2.72	2.87	3.00	3.06	3.03	3.02	17.70
Totals	16.00	17.40	17.53	16.31	15.68	17.06	99.98

The numbers are percentages of the sum of the velocities at the 42 meter points in the race. Thus, in Vertical 1, Depth 1, the velocity is 2.16% of the sum of all the velocities at the meter points.

TABLE 4.—Relative Distribution of Flow in East Race of Unit No. 3 During Tests of July and August. Water Surface at Elevation 162.85.

Depths.	Verticals.						Totals.
	1	2	3	4	5	6	
1....................	1.72	1.69	1.66	1.85	2.16	2.24	11.30
2....................	2.02	2.04	2.00	2.15	2.24	2.24	12.70
3....................	2.42	2.31	2.19	2.20	2.31	2.29	13.70
4....................	2.62	2.54	2.26	2.12	2.17	2.45	14.20
5....................	2.59	2.41	2.18	2.16	2.13	2.43	13.90
6....................	2.73	2.49	2.80	2.82	2.59	2.72	16.20
7....................	3.00	3.04	3.11	3.02	2.37	2.88	18.00
Totals..............	17.12	16.55	16.21	16.33	16.57	17.22	100.00

The percentages of Tables 3 and 4 are computed by dividing the velocity at any meter point by the sum of the velocities at all the meter points.

been compiled as average percentages from a number of such sets of observations.

In verification of Conclusion No. 4, it may be seen that each pair of velocities determined at the same point by either meter consists practically of two equal velocities. This truth has been found to apply throughout the turbine tests.

Attention should be directed to the fact that while water discharged from a turbine will be much quieted by stilling racks, yet the latter themselves are sources of disturbance, and that the violent agitation caused by the turbines is merely supplanted by an entirely different type of disturbance produced by the racks. The effect of the racks is to render the flow parallel to the side-walls of the race, but the body of the water is full of small eddies and boilers of more or less violence. The parallelism of flow at each meter point was tested by a kind of inverted weather vane, which the writer understands was first used by John R. Freeman, M. Am. Soc. C. E. The indications of parallelism of flow by this vane were extremely satisfactory at all points.

The meter section was about 30 ft. down stream from the racks. The effect of the disturbance from the racks, however, was plainly indicated by the divergence of the velocity records, based on the still-water ratings, when the two meters were run in quick succession at any meter point. It is well to note here that the velocity was greater near the bottom of the race than at the top, and that the agitation

by the racks was greater near the top than at the bottom, where the flow was extremely steady and subject to little or no agitation. This is clearly shown by the records, and especially by the fact that the clicking of the register was almost absolutely uniform when the meters were near the bottom. It is also shown by the fact that there was always a greater disparity between the meter records near the surface than at the bottom.

To be precise, the corrections for the meters should be computed for each of the 84 meter points in the races. Practically, the corrections may be made for the average velocities in the seven horizontals, because the velocities along any horizontal do not vary widely.

An example of the computation of corrections for the Haskell and Gurley-Price meters is given in Table 5.

TABLE 5.—Computation of Velocity Corrections.

Horizontal in which meter was run.	Velocities Based on the Still-Water Ratings.		Difference.	One seventh of difference + Haskell = true velocity.	Velocity excessive by Gurley. Percentage.	Velocity deficient by Haskell. Percentage.
	Gurley.	Haskell.				
1	3.94	3.36	0.58	3.44	14.5	2.4
2	4.46	3.90	0.56	3.98	12.0	2.0
3	4.74	4.44	0.30	4.48	5.8	0.96
4	4.85	4.62	0.23	4.65	4.3	0.72
5	5.08	4.89	0.19	4.92	3.2	0.55
6	5.68	5.55	0.13	5.57	2.0	0.34
7	6.32	6.15	0.17	6.17	2.3	0.38
Totals...	35.07	32.91	2.16	33.21		

$$\text{Average error of Gurley meter} = \frac{6}{7} \times \frac{2.16}{33.21} = 5.6 \text{ per cent.}$$

$$\text{Average error of Haskell meter} = \frac{1}{7} \times \frac{2.16}{33.21} = 0.93 \text{ `` ``}$$

The method given in Table 5 for computing the corrections for the velocities is based solely on the current meter ratings and the rocking experiments in conjunction with the fundamental propositions before stated. It is desirable, therefore, to have independent means of checking the methods and the numerical results.

Accordingly, a Pitot tube was operated at the meter points, in Depths 2, 3, 5, 6, and 7, simultaneously with the Haskell meter. While the meter was recording in Vertical 3 the Pitot was recording in Vertical 2. After the record was taken, both the meter and Pitot were

advanced to the next succeeding verticals, so that the meter would then record in Vertical 4 while the Pitot was in Vertical 3. Depths 1 and 4 were omitted because the tail of the Pitot was out of water in Depth 1, and the construction of the piping of the Pitot did not conform with the staging for the observers in such a way as to admit of readings in Depth 4. This might have been remedied, but no great importance was attached to these facts.

The results of these tests are given in Table 6.

It should be noticed in Table 6 that the velocities given in Verticals 1 and 6 were not taken in quick succession. The meter and tube were traversed across each horizontal, beginning with the Pitot tube in Vertical 1 and the meter in Vertical 2, and ending with the Pitot tube in Vertical 5 and the meter in Vertical 6. After the five horizontals were traversed in this manner, the Pitot tube and meter were interchanged in relative position, and readings were taken at the five depths with the Pitot tube in Vertical 6 and the meter in Vertical 1. All other readings by the meter or Pitot tube were followed immediately by a reading of the other instrument.

The writer does not wish to make too much of these Pitot tube tests. The experimenters were not familiar with the instrument, as they had used it on only one other occasion. The dynamic and static columns were drawn up on a scale graduated to single tenths and half tenths of a foot and the differences in head were merely estimated to the nearest hundredth of a foot. This may seem crude, but when it is considered that the water columns are both vibrating, sometimes violently, that twenty or thirty readings are taken for each velocity, and that all percentage errors are nearly split in two by the process of extracting the square root of the head, it will be seen that the method is probably as accurate as need be for the purpose at hand. Indeed, the results seem to justify this conclusion.

Attention may be directed to three general facts shown by the summary of Table 6. The ratios for the verticals increase toward the right in the west race. They increase toward the left in the east race. The ratios for the horizontals increase upward in both races. On the whole, the tendency is for the ratio of the Pitot velocity to the meter velocity to increase upward, as it should, as it has been shown that the meter is retarded toward the surface.

It is not wholly clear to the writer, however, why the Pitot tube

TABLE 6.—COMPARISONS BETWEEN VELOCITIES DETERMINED BY PITOT TUBE AND HASKELL METER AT VARIOUS POINTS IN THE TAIL-RACES OF UNIT NO. 5.

Depth.		VERTICALS. 1 Pitot.	1 Meter.	2 Pitot.	2 Meter.	3 Pitot.	3 Meter.	4 Pitot.	4 Meter.	5 Pitot.	5 Meter.	6 Pitot.	6 Meter.	TOTALS. Pitot.	TOTALS. Meter.	Ratio.
2........	*A*	....	...	4.06	4.22	3.84	3.29	3.76	3.33	3.86	3.63	....	....	15.52	14.47	1.072
	A	....	....	4.15	4.09	3.86	3.11	3.49	3.28	3.65	3.22	....	....	15.15	13.80	1.098
	B	....	....	3.70	3.34	3.92	3.43	4.19	3.80	4.40	4.40	....	....	16.21	14.97	1.082
	B	....	....	3.84	3.54	3.83	3.77	4.32	4.30	4.40	4.48	....	....	16.39	16.09	1.020
	C	2.56	2.49	2.34	2.34	2.30	2.28	2.08	2.21	2.32	2.17	1.68	1.77	13.28	13.26	1.001
		2.56	2.49	18.09	17.53	17.75	15.88	17.84	17.02	18.63	17.90	1.68	1.77	76.55	72.59	1.055
3........	*A*	....	....	4.22	4.21	4.14	4.02	4.05	3.98	4.35	4.20	....	....	16.76	16.41	1.020
	A	....	....	4.35	4.19	4.11	4.12	4.15	3.95	4.40	4.20	....	....	17.01	16.46	1.033
	B	....	....	4.06	4.10	4.29	4.44	4.31	4.36	4.46	4.53	....	....	17.12	17.43	0.983
	B	....	....	4.04	4.21	4.37	4.41	4.40	4.35	4.45	4.43	....	....	17.26	17.40	0.991
	C	2.58	2.71	2.36	2.39	2.34	2.41	2.15	2.21	2.36	2.50	2.09	2.06	13.88	14.28	0.972
		2.58	2.71	19.03	19.10	19.25	19.40	19.06	18.85	20.02	19.86	2.09	2.06	82.03	81.98	1.001
5........	*A*	....	....	5.26	5.41	4.57	4.67	4.37	4.41	4.32	4.32	....	....	18.52	18.81	0.984
	A	....	....	5.27	5.37	4.66	4.89	4.48	4.47	4.33	4.39	....	....	18.74	19.12	0.980
	B	....	....	4.31	4.13	4.02	4.00	4.06	4.10	4.39	4.44	....	....	16.78	16.67	1.006
	B	...	....	4.43	4.14	4.08	3.95	4.11	4.00	4.34	4.33	....	....	16.96	16.42	1.032
	C	2.61	2.69	2.91	2.89	2.48	2.53	2.61	2.64	2.34	2.46	2.16	2.14	15.11	15.35	0.985
		2.61	2.69	22.18	21.94	19.81	20.04	19.63	19.62	19.72	19.94	2.16	2.14	86.11	86.37	0.998
6........	*A*	....	....	5.36	5.61	5.47	5.55	5.41	5.55	4.93	4.91	....	....	21.17	21.62	0.978
	A	....	....	5.46	5.57	5.47	5.70	5.53	5.62	4.95	5.06	....	....	21.41	21.95	0.976
	C	2.74	2.80	2.98	3.07	3.07	3.10	2.82	2.89	2.60	2.57	2.32	2.17	16.53	16.60	0.996
		2.74	2.80	13.80	14.25	14.01	14.35	13.76	14.06	12.48	12.54	2.32	2.17	59.11	60.17	0.982

REMARKS: *A*—tests in W. race at 0.7 gate; *B*—tests in E. race at 0.7 gate; *C*—tests in W. race at 0.37 gate.

TABLE 6.—(*Continued.*)

$\frac{5}{8}$ Depth.		Verticals. 1 Pitot.	1 Meter.	2 Pitot.	2 Meter.	3 Pitot.	3 Meter.	4 Pitot.	4 Meter.	5 Pitot.	5 Meter.	6 Pitot.	6 Meter.	Totals. Pitot.	Totals. Meter.	Ratio.
7........	*A*	....	....	5.48	5.81	6.05	6.32	6.13	6.19	5.60	5.79	....	....	23.26	24.11	0.965
	A	....	....	5.56	5.79	6.16	6.32	6.21	6.03	5.66	5.79	....	....	23.59	23.93	0.985
	C	3.11	3.07	3.12	3.15	3.28	3.32	3.20	3.16	2.86	2.90	....	....	18.32	18.23	1.005
	C	3.03	3.07	3.06	3.20	3.24	3.33	3.24	3.19	2.99	3.00	2.75	2.63	15.56	15.79	0.985
		6.14	6.14	17.22	17.95	18.73	19.29	18.78	18.57	17.11	17.48	2.75	2.63	80.73	82.06	0.984
Totals..	*A*	....	....	24.38	25.26	24.07	23.85	23.72	23.46	23.06	22.85	....	....	95.23	95.45	0.998
	A	....	....	24.79	25.01	24.26	24.14	23.86	23.45	22.99	22.66	....	....	95.90	95.26	1.008
	B	....	....	12.07	11.57	12.23	11.87	12.56	12.26	13.25	13.37	....	....	50.11	49.07	1.021
	B	....	....	12.31	11.89	12.28	12.13	12.83	12.65	13.19	13.24	....	....	50.61	49.91	1.014
	C	16.63	16.83	16.77	17.04	16.71	16.97	16.10	16.30	15.47	15.60	11.00	10.77	92.68	93.51	0.991
		16.63	16.83	90.32	90.77	89.55	88.96	89.07	88.12	87.96	87.72	11.00	10.77	384.53	383.17	1.004
Ratio...	*A*	......		0.965		1.010		1.010		1.010		......		......		
	A	......		0.990		1.006		1.018		1.018		......		......		
	B	......		1.042		1.030		1.025		0.991		......		......		
	B	......		1.036		1.012		1.015		0.995		......		......		
	C	.....		0.985		0.985		0.988		0.993		......		......		
		0.988		0.995		1.007		1.011		1.003		1.021		1.004		

Remarks: *A*—tests in W. race at 0.7 gate; *B*—tests in E. race at 0.7 gate; *C*—tests in W. race at 0.37 gate.

velocity should be in the neighborhood of $1\frac{1}{2}\%$ below the meter velocity at the greater depths. It is not improbable that this would have been partly compensated for, had the observations in the east race been continued to Depths 5 and 6, but probably not entirely. In fact, there seems to be a reciprocal relation, as noted above, between the readings in the two races. This would follow naturally, owing to the symmetry of the flow in the races as to the central dividing wall, and the fact that the meters always rotate in the same direction. Thus, if there is an eddy clockwise at a certain position in either race, there should be a symmetrically located eddy counter-clockwise in the other race, and these might have opposite effects on the meter, the Pitot tube not being affected in the same way.

It is not impossible that the Haskell meter was running a trifle faster near the bottom of the race than the actual still-water ratings would indicate that it should. This would result from the fact that the rating boat was not a rigid support, nor was the water absolutely still during the ratings. This would necessitate shifting the rate line of the Haskell meter toward its ideal position of a maximum number of revolutions. This, in turn, would diminish all discharge records based on still-water ratings. On the other hand, to be consistent, it would follow that the rate line of the Gurley-Price meter would have to be shifted toward the maker's rate line, which would make for a larger correction to be added to velocities by the Haskell meter when based on still-water ratings, thus leaving true velocities in Table 5 practically unchanged. In corresponding manner, all the ratios of Tables 6 and 7 would be increased, while the meter velocities would be less in the same proportion, the effect of which would be to leave the corrected true velocities corresponding to the tables unaltered, they being in fact those of the Pitot tube. Thus, discharge estimates would not be affected materially in either case. The most reasonable explanation, however, is that the Pitot tube itself is variously affected by small amounts in different parts of the raceways.

The object, however, is not to speculate on such matters, but simply to check the computation of a small correction to the velocity, which correction may be in error by 50% and yet not affect the discharge measurements by the meter by more than three or four tenths of 1 per cent.

Interpolating a percentage for Depth 1, which was not observed in

the tests, equal to the mean given for Depth 2, 1.055, and another in Depth 4, taking an average between the means for Depths 3 and 5, practically 1.000, there may be arranged the figures in Table 7.

TABLE 7.—AVERAGE CORRECTION FOR HASKELL METER, BASED ON COMPARISON OF CURRENT METER AND PITOT TUBE RECORDS.

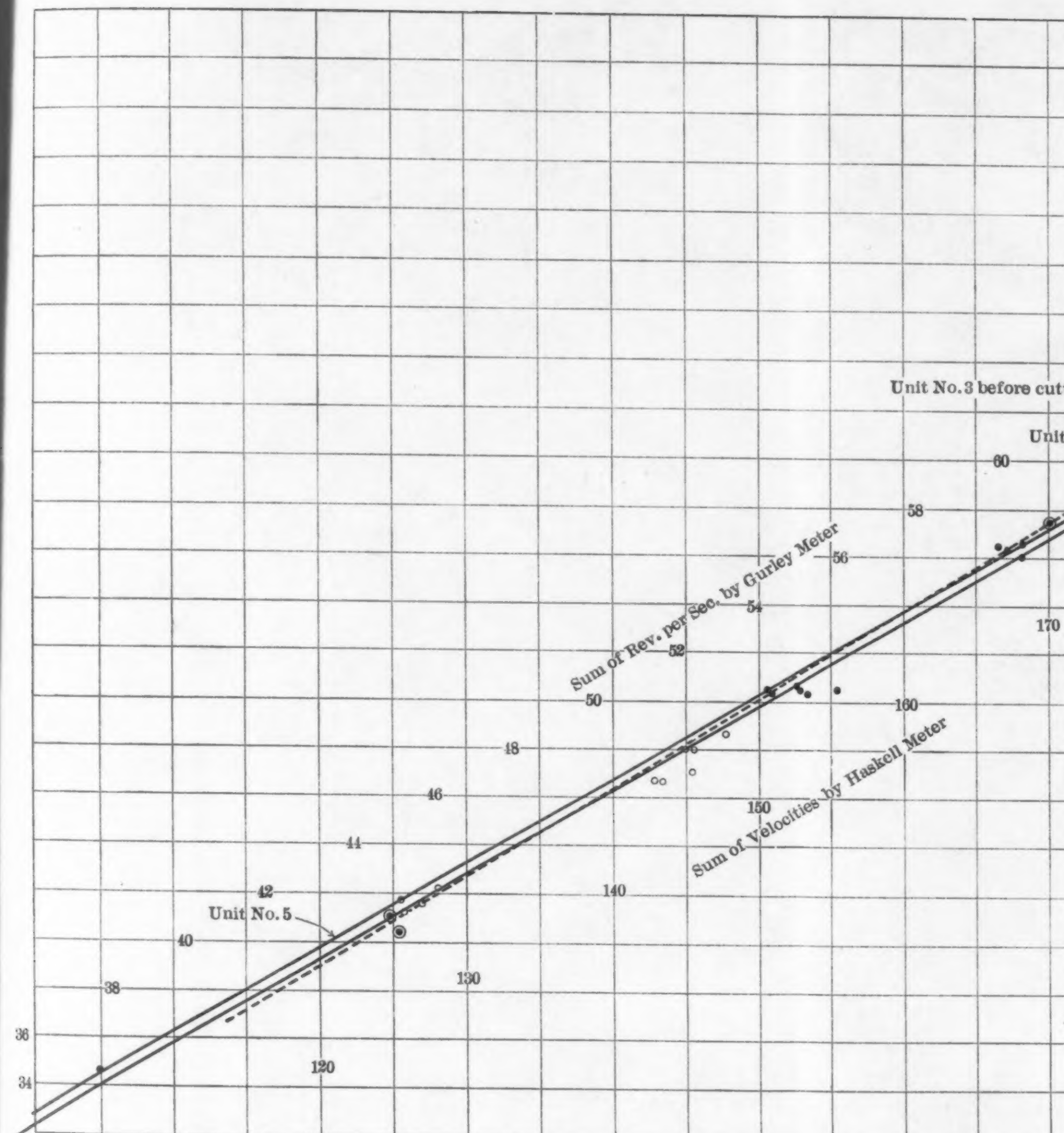
Depths.	Ratio of velocity by Pitot tube to that by current meter, from Table 6.	Relative velocity in each horizontal, being a mean for Haskell meter.	Corrected relative meter velocity.
1.....	*1.055	10.20	10.76
2.....	1.055	11.86	12.51
3.....	1.001	13.50	13.51
4.....	*1.000	14.04	14.04
5.....	0.998	14.86	14.84
6.....	0.982	16.86	16.56
7.....	0.984	18.68	18.39
		100.00	100.61

* Interpolated velocity ratio.

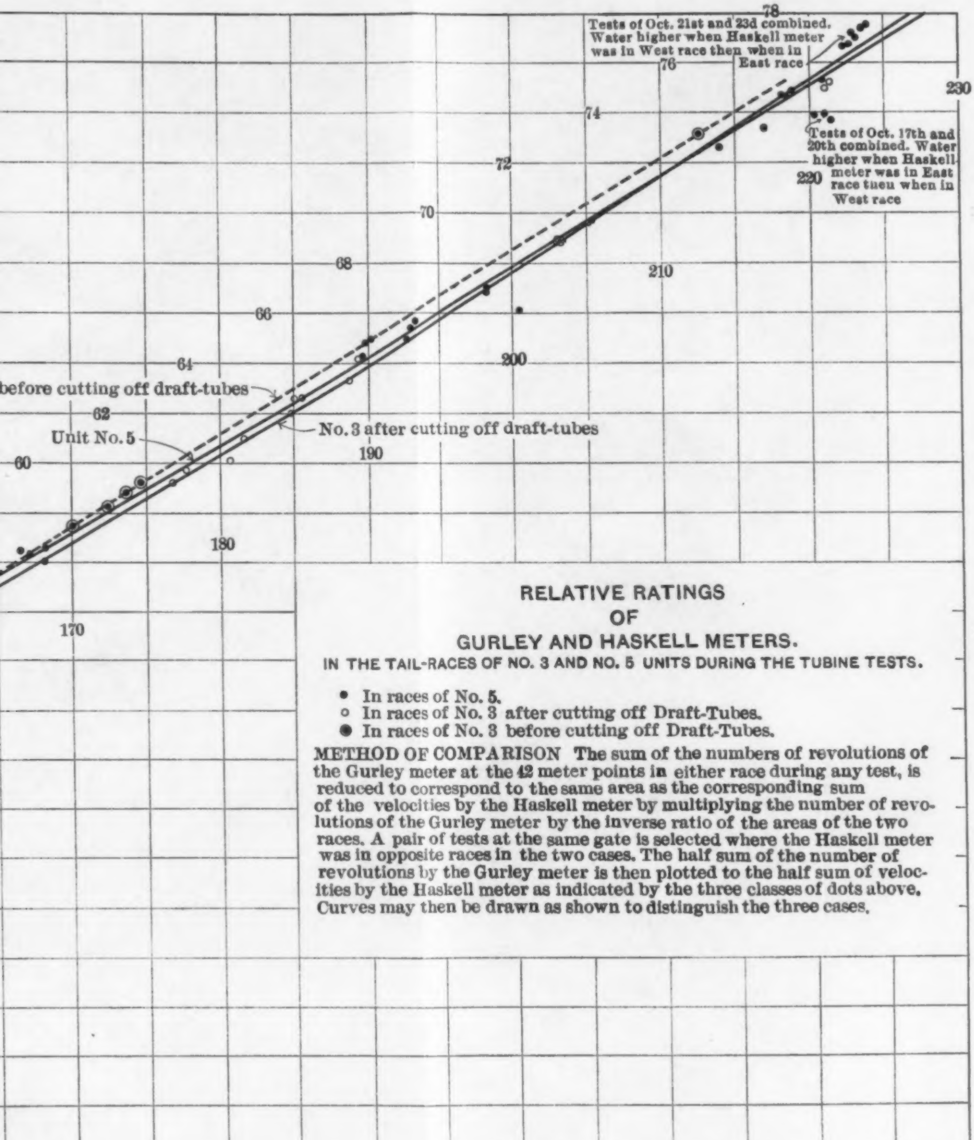
It follows from Table 7 that an approximate correction of 0.61% should be applied to discharges determined by the Haskell meter when used in these tail-races, and basing velocities on the still-water rating.

As a matter of fact, no direct use whatever was made of the still-water rating curves of the Gurley-Price meter, except to compute the corrections for the meters, which seem to be substantially verified by the Pitot tube tests. In reality, the Gurley-Price meter was rated relatively to the Haskell by comparisons of the simultaneous records taken by the meters in the two races during the numerous turbine tests.

While the Haskell meter runs in the west race, the Gurley-Price meter runs in the east race, and *vice versa.* If a pair of turbine tests be selected where the conditions are substantially the same, the Haskell meter being in the west race during one of the tests and in the east race during the other test, the total sum of the 84 velocities by the Haskell meter may be plotted to the total sum of the 84 numbers of revolutions per second by the Gurley-Price meter. If a number of such points be plotted, the curve drawn through them will represent the functional relation between the sum of the 84 velocities by the Haskell meter and the corresponding total sum of the numbers of revolutions per second by the Gurley-Price meter at the 84 meter points in the two races.



Unit No.3 before cut
Unit
60
58
56
54
52
50
48
46
44
42
40
38
36
34
Sum of Rev. per Sec. by Gurley Meter
Sum of Velocities by Haskell Meter
Unit No. 5
110
120
130
140
150
160
170



RELATIVE RATINGS OF GURLEY AND HASKELL METERS.

IN THE TAIL-RACES OF NO. 3 AND NO. 5 UNITS DURING THE TUBINE TESTS.

- • In races of No. 5.
- ○ In races of No. 3 after cutting off Draft-Tubes.
- ◉ In races of No. 3 before cutting off Draft-Tubes.

METHOD OF COMPARISON The sum of the numbers of revolutions of the Gurley meter at the 42 meter points in either race during any test, is reduced to correspond to the same area as the corresponding sum of the velocities by the Haskell meter by multiplying the number of revolutions of the Gurley meter by the inverse ratio of the areas of the two races. A pair of tests at the same gate is selected where the Haskell meter was in opposite races in the two cases. The half sum of the number of revolutions by the Gurley meter is then plotted to the half sum of velocities by the Haskell meter as indicated by the three classes of dots above. Curves may then be drawn as shown to distinguish the three cases.

As the two races are so nearly alike, it may be accepted that, on the average, half the total sum of the 84 velocities will correspond to half the total number of revolutions per second at the 84 meter points, as a functional relation between the total sum of the velocities at the 42 meter points and the corresponding total sum of the revolutions per second at these points for either race taken separately.

Plate CXXXVII is such a relative rating curve, compiled from selected turbine tests, and may be used to reduce the records of the Gurley-Price meter to what they would have been by the Haskell meter. Thus, if in any tests on Unit No. 5 the Gurley-Price meter makes a total sum of revolutions per second of 50, then, by this curve, the Haskell meter would have made a record indicating a sum of 149.3 ft. per sec.

It was not found necessary to have any regard for the individual velocity by the meter at any point, except as a part of the total sum. A system of constants and ratios was devised, so that, to obtain the mean velocity in a raceway during any tests, it was merely necessary to multiply the sum of the velocities at the 42 meter points by a factor obtained from a curve. This, however, is a matter which the writer hopes to take up more in detail at another time, and therefore, he will confine himself here to the performance of the meters in the tail-races.

With this relative rating curve and the percentages of Tables 3 and 4, it would be quite possible to determine the absolute rating curve of either meter at each of the 84 meter points; but this is hardly necessary. It will suffice here, as a matter of interest, simply to conclude this paper by determining what the average absolute rating curve of the Gurley-Price meter was in the turbine tests. This may be done by dividing the numerical value of the abscissas and ordinates of Plate CXXXVII by 42 and plotting the resulting pairs of reduced co-ordinates to the corresponding reference lines in Plate CXXXVI.

It may there be seen that the number of revolutions per second by the Gurley-Price meter in the tail-races was, on the average, about 6% greater than the still-water rating would indicate. In other words, if the writer had relied on this meter alone, without any other guide than the still-water rating curve, his discharge results would undoubtedly have been 6% too high.

These conclusions are not to be taken as casting any reflections on the Gurley-Price meter, which according to the writer's experience, is an admirably constructed instrument. In fact, the results could not have been obtained without it. The rating of the meter was perfectly definite at each of the 84 meter points in the races, but could not be determined directly by means of the still-water rating, which, in reality, is the only thing at fault. The still-water rating should never be applied without regard to the probable deviations of the speed of revolution due to disturbances in the water. An excellent check on the meters is to rate them where operated with a Pitot tube.

ADDENDA.

Since the foregoing was written, the writer has had a very instructive experience with several types of meter used simultaneously on a complicated network of mountain streams. The results of oscillating the meters have confirmed the general conclusion that in all cases, relatively, cups are accelerated considerably, while screws are retarded slightly, in turbulent water. The errors of the cup meter, based on still-water ratings, were from 3 to 6 times greater than those of the screw, and in the contrary sense.

Design of Meters.—The principal desideratum of a current meter is that it gives the resolved component of velocity in a direction fixed relatively to the meter. The fact that a cup over-registers in turbulent water while a screw under-registers, gives a basis of design which may be used to produce a meter possessing this characteristic more or less rigidly. If the blades of a screw are "cupped" to the proper extent, and in the right sense, the effect may thus be toward neutralizing the retardation which the screw would otherwise suffer.

There are other ways of producing such a meter. Professors Greene and Moody, of Rensselaer Polytechnic Institute, have conducted a series of experiments there which resulted in the production of a meter practically giving only the resolved components of velocity.

A conclusion to be drawn from these experiences is that a tail, or rudder, is a useless appendage to a meter used in stream gauging. The meter should be held rigidly in the stream or conduit, giving only the component of velocity perpendicular to the cross-section. A cup meter would give better results in turbulent water if it had no tail, but was simply allowed to run at the end of a vertical rod like an inverted cup anemometer.

The recording device for a meter designed to be held rigidly in the current should register positively for down-stream current and negatively for up-stream current. There are cases where the current beneath the surface is up-stream and the old forms of meter make no distinction, as they should.

Accuracy of Velocity Determinations by Current Meters.—Under given conditions, the current meter is a very accurate instrument. To prove this, it is only necessary to take a series of observations at a given meter point, or better, at a given set of meter points, when conditions at the gauging section are constant. Repetitions of these observations will give results checking within 1 per cent.

It is not even necessary to take observations under constant conditions, as is shown by the writer's plottings of the sums of the metered velocities at selected sets of meter points at various gauging stations. These curves are drawn through the plottings of points the co-ordinates of which are the elevation of the water surface at the given gauging station and the sum of the velocities at a selected number (25 to 100) of meter points in the cross-section. In shallow streams the meter points should be at mid-depth.

Even in the case of shallow, turbulent, stony streams, such plotted points all lie within 1% of the finally determined curve, thus showing that the performance of the meter, be it of screw or cup type, is uniform and reliable.

The main sources of error are in the application of the still-water rating, the determination of the section area, and the determination of the distribution of flow through the cross-section. The errors due to an erroneous determination of the distribution of velocities are frequently styled the "errors due to the method of calculation."

Distribution Factor.—If the discharge and section area are determined accurately for any one gauging, the mean velocity in the section may be divided by the sum of the velocities at a selected set of meter points. This ratio may be called the distribution factor for the corresponding stage of the water. In this way, corresponding values of the distribution factor and stage of the river may be determined and plotted, the co-ordinates being the elevation of the water surface and corresponding value of the distribution factor. If the work has been good, the points will plot as close to a smooth curve as the plotting of points representing the sums of velocities at the selected set of meter points.

Section Area.—The section area, next to the error in applying the still-water rating, is probably the most uncertain element of a discharge determination, where the stream is shallow and stony. Nevertheless, it has been ascertained that the surface profile of such streams at reasonably good stations is a function of the stage of the water, in many cases being simply a horizontal line. By taking a minutely accurate profile of the bottom of such a stream along the cross-section, and a number of surface profiles at various stages, the section area may be plotted to the corresponding elevation of water surface. This is another element for the accurate determination of a discharge curve.

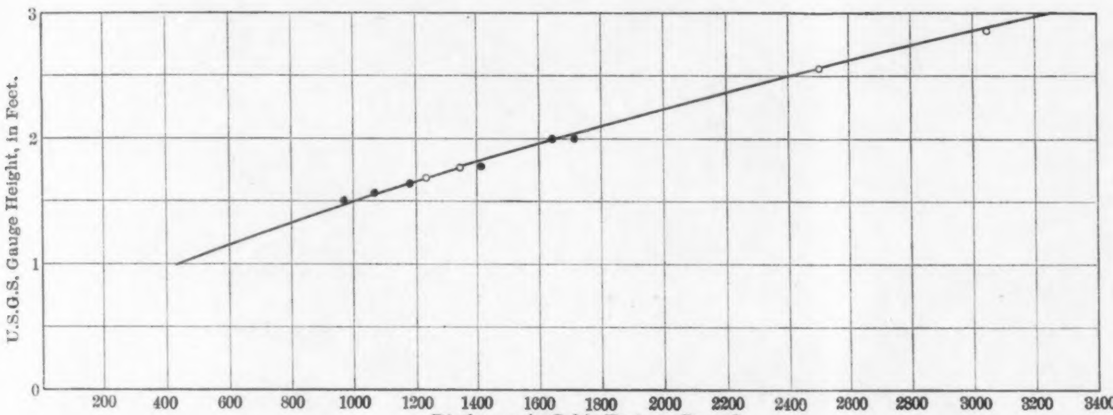
Statistical Synthesis of a Discharge Curve by Means of its Generating Elements.—According to the definitions given above, there are three principal generating elements of a discharge curve, all of which may be exhibited as curves plotted to the elevation of the water surface at the cross-section. These generating elements are, respectively, the sum of the velocities at a selected set of meter points, the distribution factor, and the area of cross-section.

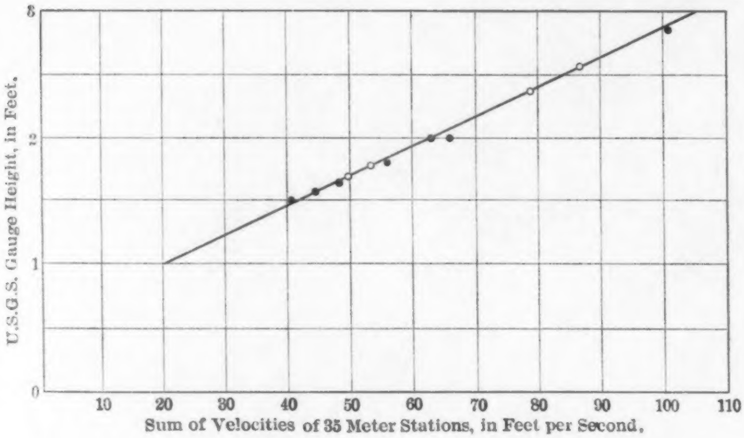
If, for any given stage of water, the corresponding values taken from the curve of sums of velocities and the curve of distribution factors be multiplied together, the result is the mean velocity in the section for that stage. The product of this mean velocity and the section area taken from the area curve for the same stage is the discharge for that stage. In this way the discharges of the stream may be computed for all stages and plotted as a finally determined discharge curve.

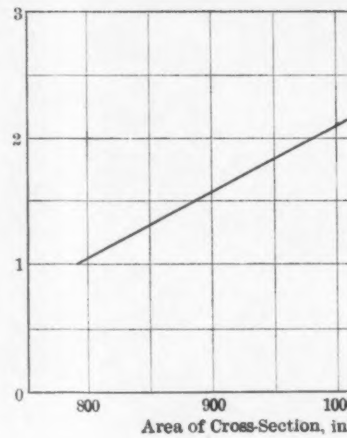
The method is as strictly statistical as though the laws of probability had been applied formally, while the result is practically as satisfactory and infinitely easier to obtain. Moreover, errors are immediately detected and located.

Good work on the mountain streams of North Carolina and Tennessee has shown that individual discharge determinations based on the foregoing processes will plot within 1% or 2%, as extreme errors, of the final discharge curve.

Distribution of Flow.—The distribution of flow is determined in the ordinary manner by vertical velocity curves. It is best, however, to be systematic. A good system is to determine the ratio of the mid-depth velocity to the mean velocity in each vertical for the whole range


U.S.G.S. Gauge Height, in Feet.
0
1
2
3
200
400
600
800
1000
1200
1400
1600
1800
2000
2200
2400
2600
2800
3000
3200
Discharge, in Cubic Feet per Second.

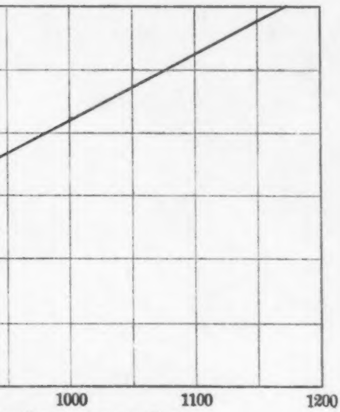

U.S.G.S. Gauge Height, in Feet.
0
1
2
3
10
20
30
40
50
60
70
80
90
100
110
Sum of Velocities of 35 Meter Stations, in Feet per Second.

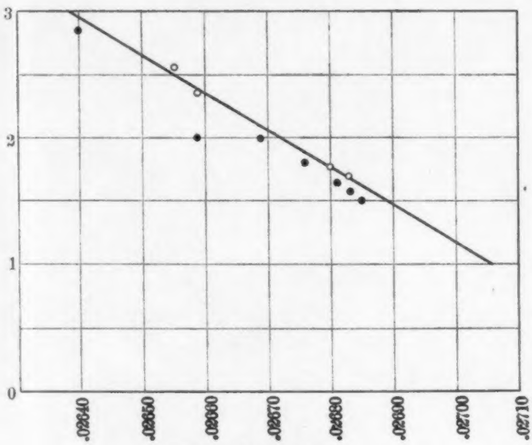

0
1
2
3
800
900

DISCHARGE CURVE FOR TUCKASEGEE RIVER AT BRYSON, N. C.

- ● Observations with Gurley Meter
- o Observations with Haskell Meter

The discharge curve has been computed from the elementary curves obtained from observations with the Haskell Meter.
The points spotted on the discharge curve indicate the individual discharge obtained from each gauging.





of stages of the river. The writer has used an average value of these ratios for each stage of the river, by which the mean mid-depth velocity for the corresponding stage is to be divided, in order to obtain the mean velocity in the section before computing a discharge. In practice, a curve may be drawn giving the mean value of the ratio for each stage of the river. The determination of this ratio is a different thing from a gauging for discharge.

Discharge Gauging.—As described above, the writer prefers to take gaugings of shallow rivers at mid-depth at equidistant stations along the cross-section. The individual discharge is then computed by dividing the sum of the products of the corrected mid-depth velocities and their corresponding areas by the average value of the before-mentioned ratio for the proper stage. The discharges at the ends of the section, below obstructions in the stream and around and through piers of bridges, should be treated separately under the title "end discharges" or "additional discharges." This additional discharge may be plotted as a function of the stage of the river. Such a plotting shows whether the calculation has been made on a consistent basis by forming a well-determined curve. Otherwise, the computer has varied his method of attack inconsistently.

Error in Using Average Ratios.—Undoubtedly, an error is made in using an average ratio of any sort, unless that average be determined properly. Thus a time average and a space average of a quantity are two entirely different things. It is important, therefore, to use the correct form of averages. However, if the values of the ratio of mid-depth velocity to mean velocity in thirty or forty verticals at a station of fairly uniform depth and velocity all lie between 1.10 and 1.18 as extremes, with an arithmetical average of about 1.14, no serious error can result in using such an average value. In fact, in this particular case, a detailed computation and one based on the average value of 1.14 differed by only one-seventh of 1 per cent.

Room for Research.—In the writer's opinion, it would pay scientific investigators to develop a meter which under all conditions would give the resolved component of velocity in a given direction, rather than to attempt to determine any functional relation between the deviations of cup and screw meters from their still-water ratings. In the foregoing studies of the results at Massena it was thought sufficiently accurate to consider the error of the cup meter about six

times the error, in the contrary sense, of the screw meter. This ratio is undoubtedly a variable, but no serious error can result by assuming it to be constant where the extremes observed would make less than 1% difference in the discharge, as was the case in the turbine tests at Massena.

Practical Application of the Methods.—Table 8 exhibits the results of a number of discharge determinations at a station where the turbulence of flow is at a minimum. In fact, the disparity between the records of the Haskell and Gurley meters is less in these observations than for those at any of the other stations operated in North Carolina and Tennessee. The average ratio of mid-depth velocity to mean velocity for 45 vertical velocity curves is 1.055. The curve has not been drawn.

TABLE 8.—Results of a Number of Discharge Observations Taken at the Station at the Wagon Bridge, Bryson, N. C.

Meter.	Gauge height.	*Sum of velocities in 35 verticals.	Section area.	Distribution factor.	Discharge.	Mean velocity.
Gurley	1.50	40.59	885.0	0.02685	965	1.090
Gurley	1.57	44.31	898.3	0.02684	1 068	1.189
Gurley	1.64	48.19	911.7	0.02681	1 179	1.292
Haskell	1.69	49.83	921.2	0.02683	1 230	1.337
Haskell	1.77	53.28	936.4	0.02680	1 338	1.429
Gurley	1.80	55.97	942.1	0.02676	1 410	1.498
Gurley	2.00	65.73	980.2	0.02659	1 713	1.748
Gurley	2.00	62.94	980.2	0.02669	1 646	1.680
Haskell	2.37	78.96	1 050.7	0.02659	2 208	2.100
Haskell	2.56	86.66	1 087.3	0.02655	2 502	2.301
Gurley	2.85	100.73	1 144.3	0.02640	3 044	2.660

*The velocities are here based solely on the still-water ratings of the meters.

Plate CXXXVIII shows the plottings of the main elements of the discharge observations of Table 8, according to the statistical methods described above. The discharge curve is determined by the three elementary curves, as already explained. The elementary curves are plotted from the records of the Haskell meter.

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PAPERS AND DISCUSSIONS

THE THEOREM OF THREE MOMENTS.*

By J. P. J. Williams, Assoc. M. Am. Soc. C. E.

The design of continuous beams, plate-girder draw spans, and swing-bridge trusses is based on the values of reactions and bending moments found by the use of a restricted form of the theorem of three moments, neglecting shear deflection. Theoretically, this usual restricted form applies only to beams built to conform accurately to the supports, which are assumed to be perfectly rigid and without possible settlement, and to beams with constant moments of inertia, and straight over the intermediate support. In practice, these conditions are never completely fulfilled. It is desirable, therefore, to determine the approximate value of the error introduced in such common practice. A complete general form of the theorem of three moments will be derived, and its application to a plate-girder draw span with variable moments of inertia will be made, in order to find the percentage of error introduced by the use of the usual formula. The theoretical maximum limit of the error thus introduced for such a typical case will be shown to be about 16.8% on the side of danger. Several fundamental and general relations for continuous beams will be given as the basis of the derivation, and also a direct derivation of the usual restricted form of the theorem. The subject matter will be divided into sections as follows:

Section 1.—The Continuous Beam.—Definition and Use of the Theorem of Three Moments.

Section 2.—Methods of Derivation.

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*This paper will not be presented at any meeting, but written communications on the subject are invited for publication with it in *Transactions*.

Section 1.—The Continuous Beam.—Definition and Use of the Theorem of Three Moments.

The continuous beam resting on n supports, at which positive or negative reactions are developed, presents a problem in which the reactions are statically indeterminate. As n unknown reactions are to be found, and only two fundamental equations of static equilibrium are available, the solution of the problem requires $n - 2$ additional equations. The theorem of three moments, which was developed in its original form by Clapeyron in 1857, makes it possible to write the $n - 2$ equations required, and thus solve the problem. It is based on the relation between the elastic distortion in adjacent spans caused by the bending moments in those spans.

Definition of Theorem.—The theorem of three moments is expressed as an equation giving the algebraic relation which exists between the bending moments at any three consecutive supports of a continuous beam and the loading on the two included spans.

This algebraic relation is thus seen to be independent of the loading on the other spans outside of the two adjacent spans considered. It is directly affected by any movement or settlement of the supports, and the general form contains terms which give the effect of such settlement or of nonconformity to supports before loading. The application of the theorem equation to the $n - 2$ pairs of adjacent spans of a continuous beam with n supports, will give $n - 2$ simultaneous equations, in terms of the bending moments at the supports and the load-

ing. The two end bending moments being known, there are only $n - 2$ unknown bending moments which, therefore, can be found at once either by algebraic solution or by the use of determinants. These bending moments at the supports being known, it is possible to find the reactions, also the shear and the bending moment for any span.

Section 2.—Methods of Derivation.

The theorem of three moments may be derived in either of two ways: First, the general bending-moment equation, $M = E\,I\,\frac{d^2 y}{dx^2}$, found by the common theory of flexure, can be used to find the slope, $\frac{dy}{dx}$, at the intermediate support, and the values as found from the equations for the two adjacent spans equated when the beam is straight over the intermediate support. Second, a more general fundamental relation between the end deflections for the two adjacent spans, when the beam is not straight, can be derived from the fact that the angle between the tangents at the intermediate support will remain constant, and this relation may be used in the derivation.

The first method is usually given in textbooks, and results only in the restricted form of the theorem. The second method is used by F. E. Turneaure,* Assoc. M. Am. Soc. C. E., for a straight beam with constant moment of inertia, also by W. H. Burr,† M. Am. Soc. C. E., for the perfectly general case of any curved beam. This second method will be applied here, both for the derivation of the restricted form and for the general form of the theorem. It is based directly on a general formula giving the relative vertical deflection, D_v, of any point due to bending, in terms of the bending moments along the beam. This general deflection formula will now be derived.

Section 3.—General Deflection Equations for Curved Beams and Arches.

Let Fig. 1 represent a perfectly general case of a fixed-ended arch subjected to elastic bending stress throughout its length. By assuming a very short length, $N_1\,N = n$, the value of the bending moment, M, can be considered as practically constant throughout such length and equal to its value at, say, N; also, the summations and co-

* "Modern Framed Structures," Vol. II.

† "Elasticity and Resistance of Materials."

ordinates can be taken from N_1 or N, indiscriminately. The differential amount of the deflection of any other point, O, can then be found by considering the movement of the chord, NO, due to the elastic bending in the short length, $N_1 N = n$. For the total deflection caused by elastic bending in the whole length, $N_1 O$ or NO, the summation of these differential deflections can be made. In this manner the general equations for change in angle, and for horizontal and vertical components of total deflection due to elastic bending, can now be derived.

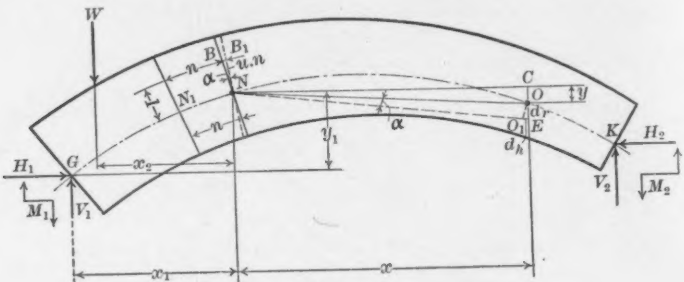


FIG. 1.

Notation.—The notation is indicated by Fig. 1, and also:

n = any small length, $N_1 N$, along a neutral axis (theoretically, a differential length; practically, often some convenient length).

M = the general value of the bending moment at any given point, N, considered as an average value for the distance, n, namely, $M = M_1 + V_1 x_1 - H_1 y_1 - \Sigma W x_2$.

α = a small angle in radians between the plane normal section, $N B$, before bending, and its position, $N B_1$, after bending has occurred in the distance, n. The slight effect of variation in distance, $N_1 N$, when the normal planes at N_1 and at N are not parallel, is negligible.

$A = \sum_O^N \alpha$ = the total angular change = the change in angle between the end tangents at N and O.

d_h and D_h = the differential and total horizontal components of deflection of O, referred to a fixed tangent at N.

d_v and D_v = the corresponding vertical components of deflection of O.

u = the rate of strain at a unit distance from the neutral axis.

f_1 = the intensity of stress at a unit distance from the neutral axis.

E = the coefficient of elasticity.

I = the moment of inertia of the normal section, $N\ B$ = the average for all normal sections in length, n.

Drawing the chord, NO, N is taken as the instantaneous center for the revolution of the point, O, due to bending, in the length, n, only, and since no bending occurs yet in the length, NO, the chord remains at a constant angle with the normal plane, NB, and revolves with that plane through the angle, α. OO_1 then represents the deflection of O due to bending in the length, n, and d_h and d_v, the two components of such deflection, are $O_1\ E$ and $O\ E$, respectively.

Now, by the definition of the rate of strain, u:

$$B\ B_1 = un;$$

and, by the common theory of flexure,

$$M = f_1\ I = u\ EI. \quad \text{Therefore, } u = \frac{M}{EI} \ldots\ldots\ldots\ldots (1)$$

From the triangle, NBB_1, as α is small:

$$\alpha = \tan.\ \alpha = u\ n \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

Therefore, from Equation (1):

$$\alpha = \frac{Mn}{EI} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

Then the total summation of these angular changes in length, ON, would be:

$$A = \sum_O^N \alpha = \sum_O^N \frac{Mn}{EI} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

Considering the similar triangles, $OO_1\ E$ and OCN:

$$\frac{O_1E}{OO_1} = \frac{OC}{ON}; \text{ therefore, } d_h = O_1E = \frac{OO_1\ OC}{ON} = \frac{OO_1}{ON}\,y.$$

$$\text{Also, } \frac{OE}{OO_1} = \frac{NC}{ON}; \text{ therefore, } d_v = OE = \frac{OO_1\ NC}{ON} = \frac{OO_1}{ON}\,x.$$

But $\frac{OO_1}{ON} = \tan.\ \alpha = \alpha = \frac{Mn}{EI}$, from Equation (3), which, when substituted in the above values for d_h and d_v, will give:

$$d_h = \frac{Mn}{EI}\,y \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

$$\text{and } d_v = \frac{Mn}{EI}\,x \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)$$

The total deflections, D_h and D_v, of the point, O, due to bending in the length, ON, therefore, will be the summation of these elementary deflections, giving:

$$D_h = \sum_{O}^{N} \frac{Mn}{EI} y \qquad (7)$$

$$D_v = \sum_{O}^{N} \frac{Mn}{EI} x \qquad (8)$$

It should be clearly understood that the values obtained by making the summations indicated in Equations (4), (7), and (8) will give the resulting effects of bending in the length, ON or ON_1, only. If bending occurs outside the portion, ON or ON_1, considered, it causes a deflection of both O and N or N_1, and affects their relative deflection with respect to each other. Therefore, Equations (7) and (8) give the relative horizontal and vertical deflections of O, with respect to N or N_1, only when the bending does not change the position of the original normal plane of reference at N or N_1. As a tangent to the neutral axis at N or N_1 is perpendicular to such normal plane, it follows that both tangent and normal plane at the end, N or N_1, of the section considered must remain fixed, if such equations are to give relative total deflections.

The origin for the co-ordinates, x and y, is at O, the point for which the deflection is being found. It is evident that this is really a moving point, as it is being deflected, but the relative amount of such deflection is insignificant in comparison with the values of x and y.

Section 4.—Application to Fixed-Ended Arches.

Perhaps the most important application of General Equations (4), (7), and (8), is to the case of the fixed-ended concrete or reinforced concrete arch, which is statically indeterminate to the third degree, there being six unknowns and only three fundamental equations of equilibrium, $\Sigma H = 0$, $\Sigma V = 0$, and $\Sigma M = 0$. Such an arch is assumed to have fixed end tangents and to sustain no relative deflections, either horizontal or vertical, between the ends, that is, the abutments are assumed to be absolutely rigid at G and K, Fig. 1. Such assumptions would make the values of Equations (4), (7), and (8), each equal to zero, when the summations are made for the total length.

G to K, giving the following three fundamental equations for elastic equilibrium, which make the problem determinate:

$$\text{From Equation (4),} \quad \sum_G^K \frac{Mn}{EI} = 0 \ldots\ldots\ldots\ldots (9)$$

$$\text{From Equation (7),} \quad \sum_G^K \frac{Mn}{EI}\, y = 0 \ldots\ldots\ldots (10)$$

$$\text{From Equation (8),} \quad \sum_G^K \frac{Mn}{EI}\, x = 0 \ldots\ldots\ldots (11)$$

It should be noted that these equations are based on deflections due to bending only, and do not account for the effect of direct thrust, shear, or temperature changes.

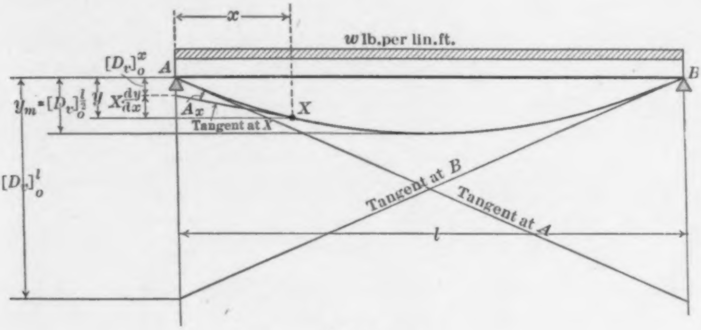


FIG. 2.

SECTION 5.—APPLICATION TO SIMPLE BEAMS.

In order to illustrate the use of Equation (8), giving the relative vertical deflection, D_v, before applying it to the case of the continuous beam, consider the simple beam uniformly loaded, as in Fig. 2. E and I will be assumed as constant and $n = dx$. Then Equation (8) becomes $D_v = \dfrac{1}{EI}\displaystyle\int_0^x Mx\,dx$, in general for the length, AX. The general value of M, for uniform load, w, per foot, is:

$$M = \frac{w}{2}\, x\,(l - x).$$

$$\text{Therefore, } [D_v]^x{}_0 = \frac{w}{2\,EI}\int_0^x (l - x)\, x^2\, dx = \frac{w}{2\,EI}\left[\frac{lx^3}{3} - \frac{x^4}{4}\right] \ldots (12)$$

If the integration is made for the half length, the maximum deflection, y_m, is obtained:

$$[D_v]_0^{\frac{l}{2}} = y_m = \frac{w}{2\,EI}\left[\frac{l^4}{24} - \frac{l^4}{64}\right]$$

or,

$$y_m = \frac{5\,wl^4}{384\,EI} \quad \text{.....................(13)}$$

If the integration is made for the whole length:

$$[D_v]_0^l = \frac{w}{2\,EI}\left[\frac{l^4}{3} - \frac{l^4}{4}\right] = \frac{wl^4}{24\,EI} \quad \text{..............(14)}$$

It is thus evident that the expression for D_v, when integrated for the total length, l, of a simple span, will not give zero (the relative deflection of the ends), but the value of the vertical movement of the end tangent at the other end, B, below the end, A. That this General Equation (8) cannot be applied to cases where vertical deflection with respect to supported ends which are not fixed is desired can be seen by considering the form of the equation. Such an equation would result by making a double integration of the fundamental deflection equation of the common theory of flexure, $\frac{d^2y}{dx^2} = \frac{M}{EI}$, omitting entirely the first constant of integration, giving $y = \int \frac{Mx\,dx}{EI} = D_v$, and also introducing x as a multiplier of M instead of some constant multiplied by x, which would result if the proper integration were made and M contained terms in x.

The second constant of integration which would now be introduced in either case is always zero, as $y = 0$ when $x = 0$. The first constant of integration is really the value of the slope, $\frac{dy}{dx}$, of the end tangent at the origin, A.

From Equation (14) it is seen that the end tangent at B has a slope, $\frac{wl^4}{24\,EI} \div l = \frac{wl^3}{24\,EI}$. This value can be checked by the ordinary equations of deflection for such a beam, found by integrating the fundamental equation, $\frac{d^2y}{dx^2} = \frac{M}{EI}$, and will be the first constant of integration. It can also be checked by using Equation (4) for change in angle, A, because, for small angles, tan. $A = A$. In general, when $n = dx$ and $E\,I$ is constant, Equation (4) gives:

$$A_x = \int_0^x \frac{Mdx}{EI} = \frac{w}{2\,EI}\int_0^x (l - x)\,x dx = \frac{w}{2\,EI}\left[\frac{lx^2}{2} - \frac{x^3}{3}\right] \ldots\ldots (15)$$

The slope of the end tangent at B with respect to the horizontal tangent at the center is then found by integrating from 0 to $\frac{l}{2}$, that is, let $x = \frac{l}{2}$ in Equation (15),

$$A = \frac{dy}{dx} = \frac{w}{2\,EI}\left[\frac{l^3}{8} - \frac{l^3}{24}\right] = \frac{wl^3}{24\,EI} \ldots\ldots\ldots\ldots (16)$$

The total deflection, y, at any point, x, is not given by the D_v Equation (12), but can be found by adding to $\left[D_v\right]_0^x$ the quantity obtained by multiplying the slope at X by x, as is seen in Fig. 2. The value of the slope, $\frac{dy}{dx}$, at X can be found either by the integration of the fundamental equation, $\frac{d^2\,y}{dx^2} = \frac{M}{EI}$, or by using Equation (12) for A_x. The latter can be shown to check the former method as follows, as A_x is the angle between the tangents at A and at X:

$$\frac{dy}{dx} \text{ at } X = \text{slope at } A - A_x = \frac{wl^3}{24\,EI} - \frac{w}{2\,EI}\left(\frac{lx^2}{2} - \frac{x^3}{3}\right)$$

$$= -\frac{w}{2\,EI}\left[\frac{lx^2}{2} - \frac{x^3}{3} - \frac{l^3}{12}\right] \ldots\ldots\ldots\ldots\ldots (17)$$

Equation (17) checks the value obtained by general integration, but with opposite sign.

The total deflection, y (using Equations (12) and (17)), therefore, is:

$$y = \left[D_v\right]_0^x + x\left(\frac{dy}{dx} \text{ at } X\right) = \frac{w}{2\,EI}\left[\frac{lx^3}{3} - \frac{x^4}{4} - \frac{lx^3}{2} + \frac{x^4}{3} + \frac{l^3\,x}{12}\right]$$

Therefore, $y = -\frac{w}{24\,EI}\left[2\,lx^3 - x^4 - l^3\,x\right] \ldots\ldots\ldots\ldots\ldots\ldots\ldots (18)$

Equation (18) checks the value obtained by general integration, but with opposite sign.

It should be explained that the difference in sign is due to the fact that the D_v Equation gives positive values for deflection when the

positive bending moment occurs, while, in the notation for deflection by the general integration of $\frac{d^2 y}{dx^2} = \frac{M}{EI}$, the deflection is downward or negative for the positive bending moment.

It is also important to observe that the origin for x in the D_v Equation is at the deflecting point, the result being positive if that point has deflected upward with respect to the tangent at the other end of the section considered. No constants of integration are introduced in the foregoing, as they are zero.

Whenever concentrated loads occur, it is necessary to make separate integrations for each segment of the beam, because the law of variation of the bending moment with x will change at each load.

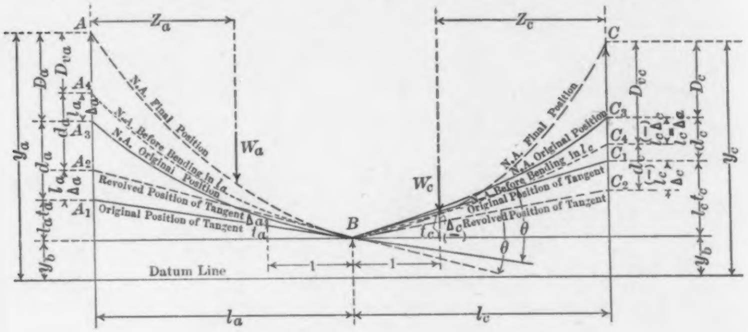


FIG. 3.

The deflection, $y_m - y$, at any point, X, cannot be obtained from the integrated value for D_v in Equation (12), because the origin for x is not at X. The bending moment, M, must first be written in terms of x_1 (say) with X as the origin, and the integration made by the General Equation, $D_v = \int \frac{Mx\,dx}{EI}$. If, however, the bending moment, M, is constant in the section considered, as, for instance, in the case of a simple beam loaded with two symmetrical concentrated loads, the deflection between the center and the load on either side can be obtained by Equation (12), because M has the same value, regardless of the origin used.

SECTION 6.—FUNDAMENTAL GENERAL RELATION FOR CONTINUOUS BEAMS.

Consider two adjacent spans, l_a and l_c, of any continuous beam with a curved neutral axis (N. A. Fig. 3) resting only on the interme-

diate support, B, before the application of loads. Neglecting the effect of deflection due to shear, a fundamental equation based on the fact that the tangents to the neutral axes at B, for the two adjacent spans, remain at a constant fixed angle with each other, may be derived as follows:

Let t_a = the tangent of the angle between the original position of the tangent at B for the span, l_a, and the horizontal.

Δ_a = the change in t_a due to the rotation of the tangent at B.

D_a = the vertical distance from the original position of the neutral axis to the support, A, + if upward.

d_a = the vertical distance at A between the original tangent and the original neutral axis.

D_{va} = the total vertical deflection of the neutral axis at A due to bending in the span, l_a, only, that is,

$$D_{va} = \sum_A^B \frac{Mnx}{EI}.$$

t_c, Δ_c, D_c, d_c, D_{vc} = the corresponding quantities for the span, l_c, as shown.

y_a, y_b, and y_c = the elevations of the supports, A, B, and C.

The vertical movement, D_a, at A, between the original and final positions of the neutral axis, is seen to be composed of two parts: First, the movement, $A_3\ A_4$, due to the rotation of the tangent at B; and second, the movement or deflection, $A_4\ A$, due to the bending in the span, l_a, with respect to the tangent at B. This rotation of the tangent at B would certainly occur in the general case. The bending deflection for the span, l_a, only, is not equal to the total movement, D_a, except for the special case of symmetry about B such that no movement of the tangents at that point would take place.

From a consideration of Fig. 3, it is seen that the vertical movement at A between the original and final positions of the tangent is $A_1\ A_2 = l_a\ \Delta_a$; and the corresponding movement of the neutral axis vertically at A is $A_3\ A_4$, and also $= l_a\ \Delta_a$, because d_a remains practically constant. The bending deflection, $D_{va} = A_4\ A$, in the span, l_a, is the other portion of the movement, $A_3\ A$, of the neutral axis to bring it to the support at A. This deflection, by Equation (8), is:

$$D_{va} = \sum_A^B \frac{Mnx}{EI}.$$

If the $+ M$ terms are greater than the $- M$ terms, D_{va} is $+$ or upward, as shown. Note that the origin is at A.

As θ is constant:

$$\Delta_a = - \Delta_c \qquad (19)$$

and, from Fig. 3,

$$D_{va} = D_a - l_a \, \Delta_a \qquad (20)$$

Also,

$$D_{vc} = D_c - l_c \, \Delta_c = D_c + l_c \, \Delta_a \qquad (21)$$

Solve Equations (20) and (21) for Δ_a, and equate:

$$\Delta_a = \frac{D_a - D_{va}}{l_a} = \frac{D_{vc} - D_c}{l_c}$$

whence,

$$\frac{D_a}{l_a} + \frac{D_c}{l_c} = \frac{D_{va}}{l_a} + \frac{D_{vc}}{l_c} \qquad (22)$$

Substituting the values of D_{va} and D_{vc}:

$$\frac{D_a}{l_a} + \frac{D_c}{l_c} = \frac{\sum_A^B \frac{Mnx}{EI}}{l_a} + \frac{\sum_C^B \frac{Mnx}{EI}}{l_c} \qquad (23)$$

Equation (23) is the fundamental and perfectly general equation by which the theorem of three moments can be developed by determining values for the summations indicated in the second member. As a check on Equations (22) and (23), consider the usual case of a horizontal beam resting on three supports, A B C. Assume a single load, W, on one span only; the final position of the neutral axis and the tangent at B would be as shown in Fig. 4. For this case, $D_a = 0$, $D_c = 0$, and Equation (22) gives:

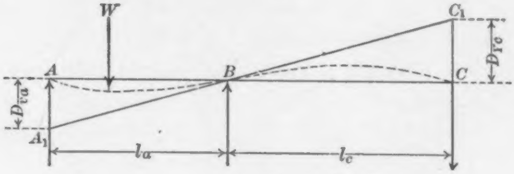
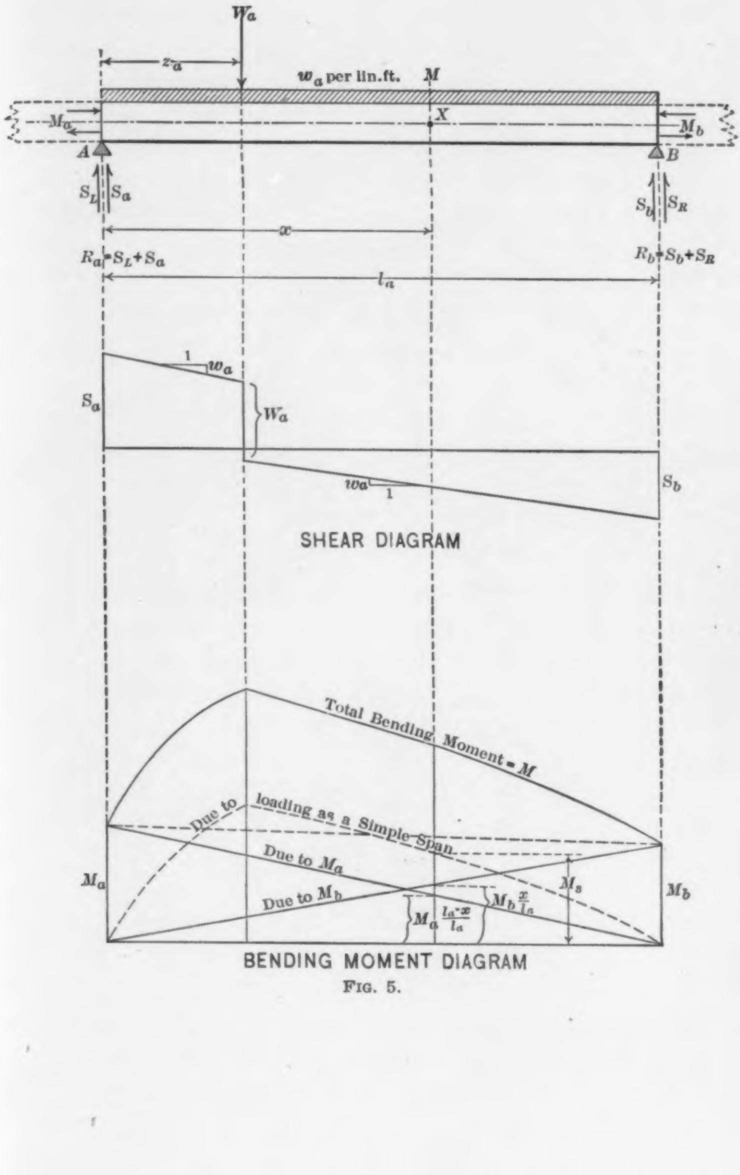


FIG. 4.

$$0 = \frac{D_{va}}{l_a} + \frac{D_{vc}}{l_c}; \text{ whence, } \frac{D_{va}}{l_a} = - \frac{D_{vc}}{l_c}.$$

This agrees with the result of considering the similar triangles, $AA_1 B$ and BCC_1, as D_{vc} would be negative.



Wa
za
wa per lin.ft.
M
X
Ma
Mb
A
B
SL
Sa
Sb
SR
x
Ra=SL+Sa
Rb=Sb+SR
la
1
wa
Sa
Wa
wa
1
Sb
SHEAR DIAGRAM
Total Bending Moment=M
Due to loading as a Simple Span
Due to Ma
Due to Mb
Ma
Mb
Ms
BENDING MOMENT DIAGRAM

FIG. 5.

Section 7.—General Values of Bending Moments, Shears, and Reactions, for Any Span of a Continuous Beam.

Consider any span, AB, of a continuous beam, Fig. 5, and let:

M_a = the bending moment at A (from the theorem of three moments).

M_b = the bending moment at B (from the theorem of three moments).

M_s = the bending moment at X, due to the loading on l_a as a simple span.

M = the total bending moment at X.

S_L = the reaction at A from the span to the left of A = — shear to the left of A.

S_a = the reaction at A from the span, l_a = shear to the right of A.

S_R = the reaction at B from the span to the right of B = shear to the right of B.

S_b = the reaction at B from the span, l_a = — shear to the left of B.

R_{sa} = the reaction at A from the span, l_a, as a simple beam.

R_{sb} = the reaction at B from the span, l_a, as a simple beam.

$R_a = S_L + S_a$ = the total reaction at A.

$R_b = S_b + S_R$ = the total reaction at B.

To find the value of S_a, for the given span, write the equation for the value of the bending moment at $B = M_b$, considering all forces acting on the span, l_a:

$$M_b = M_a + S_a\, l_a - w_a \frac{l_a^2}{2} - W_a (l_a - z_a) \qquad \text{(24)}$$

Solving for S_a:

$$S_a = \frac{M_b - M_a}{l_a} + \frac{w_a\, l_a}{2} + W_a \frac{l_a - z_a}{l_a} = \frac{M_b - M_a}{l_a} + R_{sa} \qquad \text{(25)}$$

Note that if $M_a = M_b$, $S_a = R_{sa}$.

Similarly:

$$S_b = \frac{M_a - M_b}{l_a} + R_{sb} \qquad \text{(26)}$$

Or, S_b = (the total loading on l_a) — S_a. ....(26a)

The values of M_a and M_b are first found from the theorem of three moments, and are generally negative. The sign must always be included in making substitutions in the foregoing formulas.

Having found the values of S_a and S_b for adjacent spans, the reactions are at once found by addition, noting that $S_L =$ the S_b for the span on the left, and $S_R =$ the S_a for that on the right.

The general value for M, at any point, X, can be stated in two ways:

First, $M = M_a + S_a\,x$ — (moment of loads on l_a to left of X about X)..........(27)

Second, $M = \dfrac{M_a\,(l_a - x)}{l_a} + \dfrac{M_b\,x}{l_a} + M_s$..........(28)

This second form is seen to result at once from similar triangles, in Fig. 4, and shows that M can be considered as composed of three distinct parts: (a) that due to M_a; (b) that due to M_b; and (c) that due to loads on l_a as a simple beam. The terms due to (a) and (b) can also be explained by noting that these end bending moments produce vertical couples; $+ M_a$ produces a force, $+ \dfrac{M_a}{l_a}$, at B, and $- \dfrac{M_a}{l_a}$, at A, and similarly, M_b gives a force couple, $+ \dfrac{M_b}{l_a}$, at A, and $- \dfrac{M_b}{l_a}$, at B. These forces, produced as vertical couples to balance M_a and M_b, also explain the form of Equations (25) and (26) for S_a and S_b.

Section 8.—Derivation of the Usual Restricted Form of the Theorem of Three Moments.

For the usual practical case, the beam is straight, rests on the supports, may slope at an angle, B, with the horizontal, and E and I are assumed as constant throughout the entire length. For such a case, then:

$D_a = 0$, $D_c = 0$, $n = dx$ sec. B, and Equation (23) becomes:

$$0 = \frac{\displaystyle\int_0^{l_a} \frac{M\,x\,dx \text{ sec. } B}{EI}}{l_a} + \frac{\displaystyle\int_0^{l_c} \frac{M\,x\,dx \text{ sec. } B}{EI}}{l_c}$$

As sec. B, E, and I, are constant, they can be multiplied out, and

$$\int_0^{l_a} \frac{M\,x\,dx}{l_a} + \int_0^{l_c} \frac{M\,x\,dx}{l_c} = 0 \text{..........(29)}$$

To integrate $\displaystyle\int_0^{l_a} M\,x\,dx$, substitute for M the general value from Equation (28),

$$\int_0^{l_a} M\,x\,dx = \frac{M_a}{l_a}\int_0^{l_a}(l_a - x)\,x\,dx + \frac{M_b}{l_a}\int_0^{l_a} x^2\,dx + \int_0^{l_a} M_s\,x\,dx. \quad (30)$$

The first term becomes, $\frac{M_a}{l_a}\left[\frac{l_a\,x^2}{2} - \frac{x^3}{3}\right] = \frac{M_a\,l_a^2}{6}$;

the second term becomes, $\frac{M_b}{l_a} \times \frac{l_a^3}{3} = \frac{2\,M_b\,l_a^2}{6}$.

The third term in M_s consists of three parts: First, the bending moment in the segment, z_a, due to the concentrated load, W_a, that is, $\frac{W_a\,(l_a - z_a)}{l_a}\,x$; second, the bending moment in the segment, $l_a - z_a$, due to the concentrated load, W_a, that is, $\frac{W_a\,z_a}{l_a}\,(l_a - x)$; and third, the bending moment at any point due to the uniform load, w_a, that is, $\frac{w_a\,x}{2}\,(l_a - x)$.

Using these values, the third term then becomes:

$$\int_0^{l_a} M_s\,x\,dx = \frac{W_a}{l_a}\left[\int_0^{z_a}(l_a - z_a)\,x^2\,dx + z_a\int_{z_a}^{l_a}(l_a - x)\,x\,dx\right] + \frac{w_a}{2}\int_0^{l_a}(l_a - x)\,x^2\,dx = \frac{W_a}{l_a}\left[(l_a - z_a)\,\frac{z_a^3}{3}\right.$$

$$\left. + z_a\left(l_a\,\frac{x^2}{2} - \frac{x^3}{3}\right)_{z_a}^{l_a}\right] + \frac{w_a}{2}\left[l_a\,\frac{x^3}{3} - \frac{x^4}{4}\right]_0^{l_a} = \frac{W_a}{l_a}\left[\frac{l_a\,z_a^3}{3}\right.$$

$$\left. - \frac{z_a^4}{3} + \frac{z_a\,l_a^3}{2} - \frac{z_a\,l_a^3}{3} - \frac{l_a\,z_a^3}{2} + \frac{z_a^4}{3}\right] + \frac{w_a\,l_a^4}{24}$$

$$\int_0^{l_a} M_s\,x\,dx = \frac{W_a\,z_a}{6}\,(l_a^2 - z_a^2) + \frac{w_a\,l_a^4}{24} \quad \ldots\ldots\ldots (31)$$

Substituting the foregoing values in Equation (30):

$$\int_0^{l_a} M\,x\,dx = \frac{M_a\,l_a^2 + 2\,M_b\,l_b^2 + W_a\,z_a\,(l_a^2 - z_a^2) + \frac{w_a\,l_a^4}{4}}{6} \quad \ldots (32)$$

By analogy, for the span, l_c;

$$\int_0^{l_c} M\,x\,dx = \frac{M_c\,l_c^2 + 2\,M_b\,l_c^2 + W_c\,z_c\,(l_c^2 - z_c^2) + \frac{w_c\,l_c^4}{4}}{6} \quad \ldots (34)$$

Substituting these values in Equation (29), eliminating the factor, 6, and transposing:

$$M_a\,l_a + 2\,M_b\,(l_a + l_c) + M_c\,l_c = -\,\Sigma\,W_a\,\frac{z_a}{l_a}\,(l_a^2 - z_a^2)$$

$$-\,\Sigma\,W_c\,\frac{z_c}{l_c}\,(l_c^2 - z_c^2) - \frac{w_a\,l_a^3}{4} - \frac{w_c\,l_c^3}{4} \quad \ldots . (35)$$

The terms in W_a and W_c have the summation sign introduced in order to account for all such concentrated loads on the spans. Equation (35) is the usual restricted form of the theorem of three moments, and is used in general for all cases of continuous beams, even for swing-bridge trusses, where the neutral axis is not straight and the moment of inertia is not constant.

Section 9.—Effect of Settlement of Supports.

The foregoing form of the theorem assumes that the beam just rests on rigid supports. If the supports should settle, the values, D_a and D_c, would no longer be zero, but would be equal to the amount of such relative settlement. Assuming the beam to be straight, but having a slope, B, with the horizontal, Equation (23) becomes (as EI and sec. B are constant, n being equal to dx sec. B):

$$\frac{EI}{\text{sec. } B}\left[\frac{D_a}{l_a} + \frac{D_c}{l_c}\right] = \int_0^{l_a} \frac{M\,x\,dx}{l_a} + \int_0^{l_c} \frac{M\,x\,dx}{l_c} \quad \ldots.(36)$$

The second member of this equation is the same as that in Equation (29), which has been integrated in Equation (35). Thus, the only difference between Equation (35) and this case, is that the term, $\frac{6\,EI}{\text{sec. } B}\left[\frac{D_a}{l_a} + \frac{D_c}{l_c}\right]$, must be added, the factor, 6, having been multiplied out in getting Equation (35). The equation would then be:

$$M_a\,l_a + 2\,M_b\,(l_a + l_c) + M_c\,l_c = -\,W_a\,\frac{z_a}{l_a}\,(l_a^2 - z_a^2)$$

$$-\,W_c\,\frac{z_c}{l_c}\,(l_c^2 - z_c^2) - \frac{w_a\,l_a^3}{4} - \frac{w_c\,l_c^3}{4} + \frac{6\,EI}{\text{sec. } B}\left[\frac{D_a}{l_a} + \frac{D_c}{l_c}\right] \quad ..(37)$$

This last term would be large, as it contains the multiplier, $E\,I$, and would be additive in effect if D_a and D_c were negative, that is, when the end supports settled downward with respect to the intermediate support.

All computations for bending moments, M, are made in foot-pounds or foot-kips, therefore, the last term should be put in form for such computation by dividing by 12^2, as E and I are in inches, and D_a and D_c are in feet. Therefore, the last term of Equation (37) becomes:

$$\frac{EI}{24\text{ sec. } B}\left[\frac{D_a}{l_a} + \frac{D_c}{l_c}\right] \quad \ldots\ldots\ldots\ldots\ldots\ldots.(37a)$$

For the usual case of a horizontal beam, sec. $B = 1$.

Section 10.—Derivation of the Complete General Form of the Theorem of Three Moments.

The most general form of the theorem of three moments, applicable to any curved continuous beam with a variable moment of inertia and not originally resting on all supports, can be derived directly from the fundamental Equation (23). It is only necessary to obtain a general working form for the summations appearing in the second member of that equation, as follows:

Using the general value for M, as derived for Equation (18), there results:

$$\sum_A^B \frac{M n\, x}{EI} = \sum_A^B \left[\frac{M_a}{l_a}(l_a - x) + \frac{M_b\, x}{l_a} + M_s\right]\frac{n\, x}{EI} \quad \text{..(38)}$$

Let $n = dx$ sec. β, and assume E to be constant, then:

$$\sum_A^B \frac{M n\, x}{EI} = \frac{1}{l_a E}\left[M_a \sum_A^B (l_a - x)\, x\, dx \frac{\text{sec. } \beta}{I} + M_b \sum_A^B x^2\, dx \frac{\text{sec. } \beta}{I} + l_a \sum_A^B M_s\, x\, dx \frac{\text{sec. } \beta}{I}\right] \quad \text{...... (39)}$$

By analogy, for span, l_c:

$$\sum_C^B \frac{M n\, x}{EI} = \frac{1}{l_c E}\left[M_c \sum_C^B (l_c - x)\, x\, dx \frac{\text{sec. } \beta}{I} + M_b \sum_C^B x^2\, dx \frac{\text{sec. } \beta}{I} + l_c \sum_C^B M_s\, x\, dx \frac{\text{sec. } \beta}{I}\right] \quad \text{...... (40)}$$

By substituting the foregoing values in Equation (23), the following General Equation results:

$$E\left(\frac{D_a}{l_a} + \frac{D_c}{l_c}\right) = \frac{M_a}{l^2_a} \sum_A^B (l_a - x)\, x\, dx \frac{\text{sec. } \beta}{I} + M_b\left(\frac{\sum_A^B x^2\, dx \frac{\text{sec. } \beta}{I}}{l^2_a} + \frac{\sum_C^B x^2\, dx \frac{\text{sec. } \beta}{I}}{l^2_c}\right) + \frac{M_c}{l^2_c} \sum_C^B (l_c - x)\left(x\, dx \frac{\text{sec. } \beta}{I}\right) + \frac{\sum_A^B M_s\, x\, dx \frac{\text{sec. } \beta}{I}}{l_a} + \frac{\sum_C^B M_s\, x\, dx \frac{\text{sec. } \beta}{I}}{l_c} \quad \text{..(41)}$$

This equation is the most general form of the theorem of three moments. Whenever the variables, sec. β and I, can be expressed as continuous functions of x, the summation sign can be replaced by the integration sign, and the mathematical integration can be made. Practically, it is generally found impossible to express these variables as continuous functions of x, and then make the integrations.

Usually, I is constant for short sections of the beam, in which case it is best to make successive summations for such sections, using the general form of the integration, with I constant, and making the limits correctly account for the variation in I for different sections. This method will now be illustrated by a practical example.

Section 11.—Application to Plate-Girder Draw Span with Variable Moment of Inertia.

As an example of the practical method of using the general Equation (41), and also in order to determine the effect of the variable moment of inertia, consider the following case of a plate-girder span continuous over three supports. This is the same girder as used by Johnson, Bryan, and Turneaure* to illustrate the use of a similar formula obtained by applying the principle of least work. It is used here in order to find how closely the results of the different methods agree.

Data for Girder.—$l = l_a = l_c = 60$ ft., that is, the total length $= 120$ ft. The girder is symmetrical about its center line.

The relative values of the moment of inertia, I, at the following distances from the outside ends, are:

0 to 12 ft., $I = 1$
12 to 36 ft., $I = 1.38$
36 to 48 ft., $I = 1.15$
48 to 60 ft., $I = 1.92$

The girder rests on all the supports, and has a straight horizontal neutral axis, therefore, $D_a = 0$, $D_c = 0$, sec. $\beta = 1$.

The end bending moments are zero, therefore, $M_a = 0$, $M_c = 0$.

Noting the fact of the constant, I, for the given sections, let x_1 and x_2 be the distances from the end of the girder to the beginning and the end, respectively, of a given section having a constant, I, and let $\frac{1}{I} = i$ for such section. Then, by substituting in the General Equation (41), there results, after multiplying through by l^2:

$$0 = M_b \left[\sum_A^B \int_{x_1}^{x_2} i\, x^2\, dx + \sum_C^B \int_{x_1}^{x_2} i\, x^2\, dx \right]$$
$$+ \, l \left(\sum_A^B i\, M_s x\, dx + \sum_C^B i\, M_s x\, dx \right) \ldots\ldots\ldots \quad (42)$$

* "Modern Framed Structures," Vol. II, p. 39.

For the case here considered, with a fully loaded beam carrying a uniform load, w lb. per ft., $M_s = \frac{w}{2}\, x\,(l - x)$. Therefore,

$$\sum i\, M_s\, x\, dx = \frac{w}{2} \sum \int_{x_1}^{x_2} i\, x^2\,(l - x)\, dx$$

$$= \frac{w}{2} \sum \left[\, i\, x^3 \left(\frac{l}{3} - \frac{x}{4}\right) \right]_{x_1}^{x_2}$$

Then Equation (42) will give the following value for M_b:

$$M_b = -\,\frac{3\, wl}{2}\; \frac{\sum_0^l \left[\, i\, x^3 \left(\frac{l}{3} - \frac{x}{4}\right) \right]_{x_1}^{x_2}}{\sum_0^l \left[\, i\, x^3 \right]_{x_1}^{x_2}} \quad \text{.........(43)}$$

The computation can best be made in the form shown by Table 1.

TABLE 1.—Computation for M_b.

6-ft. Sections.	(1) x: x_1	(2) x: x_2	(3) I	(4) i	(5) x^3		(6) $\left[i\, x^3\right]_{x_1}^{x_2}$ Columns 4 × 5	(6)	(7) $\frac{l}{3} - \frac{x}{4}$	(8) $\left[i x^3\left(\frac{l}{3} - \frac{x}{4}\right)\right]_{x_1}^{x_2}$ Columns 6 × 7	(8)
1-2	0	12	1.0	1.0	x_2	1 730	1 730	......	17	29 400	
					x_1	0	— 0	1 730	20	0	29 400
3-4-5-6	12	36	1.38	0.725	x_2	46 700	33 700	......	11	371 000	
					x_1	1 730	— 1 200	32 500	17	— 21 000	350 000
7-8	36	48	1.15	0.87	x_2	110 600	96 000	......	8	768 000	
					x_1	46 700	— 40 400	55 600	11	— 445 000	323 000
9-10	48	60	1.92	0.52	x_2	216 000	112 400	......	5	562 000	
					x_1	110 600	— 57 400	55 000	8	— 460 000	102 000
							Σ = 144 800			Σ = 804 400	

Then, $$M_b = -\,\frac{3}{2}\, wl \times \frac{804\,400}{144\,800} = -\,8.34\, wl.$$

For the given case, $l = 60$ ft.:

Therefore, $M_b = -8.34 \times 60\ w = -500\ w$.

When I is assumed as constant, as in the usual form of the theorem, let $l_a = l_c = l$, and $w_a = w_c = w$, in Equation (35),

$$2\ M_b\ (l + l) = -2\left(\frac{wl^3}{4}\right), \text{ whence,}$$

$$M_b = -\frac{wl^2}{8} = \frac{-w \times 60^2}{8} = -450\ w.$$

The exact method, therefore, gives a value for M_b about 11% higher than the approximate method, using the constant, I. The method of least work* gave $M_b = -492\ w$, about 2% less than the value here found by the exact method.

To Derive the Maximum Limit for Error.—The error introduced by using the general form of the theorem of three moments in such cases of girders with variable I, carrying the uniform load, and symmetrical over three supports, can be shown to be about 15% for the limiting case where I is assumed to vary directly with M, that is, for a theoretically perfect design, which, of course, is a practically impossible case.

The reaction, R_a, can be found by locating the point of contraflexure where the bending moment changes sign, and noting that the end shears for the equivalent simple beam, between points of contraflexure or zero bending moment, can be found exactly as in simple beam analysis. For the foregoing case of two equal spans loaded with equal uniform loads, the tangent at the intermediate support, B, remains fixed and horizontal. Therefore, if x_0 = distance from A to the point of contraflexure:

$$[D_{va}]^l_0 = \int_0^l \frac{M\ x\ dx}{EI} = \int_0^{x_0} \frac{M\ x\ dx}{EI} - \int_{x_0}^l \frac{M\ x\ dx}{EI} = 0.$$

Now let $\dfrac{M}{I} = K$, substitute, and cancel the constants, $\left(\dfrac{K}{E}\right)$,

$$\int_0^{x_0} x\ dx = \int_{x_0}^l x\ dx$$

$$\frac{x_0^2}{2} = \frac{l^2}{2} - \frac{x_0^2}{2}. \text{ Therefore, } x_0 = \frac{l}{\sqrt{2}} = 0.707\ l.$$

(Note that $x_0 = 0.75\ l$, when I is assumed as constant.)

Then, $R_a = \dfrac{wx_0}{2} = 0.354\ wl$; and, $M_b = R_a\ l - \dfrac{wl^2}{2} = 0.354\ wl^2 - 0.5\ wl^2 = -0.146\ wl^2$.

* As applied in "Modern Framed Structures," Vol. II, p. 40.

As shown before, the usual form of theorem, with I assumed as constant, gives for this case:

$$M_b = -\frac{wl^2}{8} = -0.125\, wl^2.$$

Therefore, the approximate method gives results which are too low by a maximum limit of 15% of the exact value, or 16.8% of the approximate value.

The general conclusion that the usual approximate formula will give M_b from 10 to 15% too low, when applied to girder spans with widely varying moments of inertia, seems to be quite justifiable, the effect of shear deflection being neglected.

Two cases of swing-bridge girders, with the variable, I, are solved by C. W. Hudson, M. Am. Soc. C. E., by the formulas derived from the method of deflections.* The first is a light single-track girder with two equal spans of 68 ft.; and the second is a heavy double-track girder with two equal spans of 78 ft. In the first case the center bending moment, M_b, is 15% greater, and in the second case 11% greater, by the exact method than by the approximate method. Mr. Hudson,† also computes the effect of shear deflection on the center bending moment, and finds, in the first case, a decrease of 2.6%, and in the second a decrease of 5%, as compared with the value obtained by neglecting shear distortion.

It is evident that the usual approximate assumption, neglecting the variable, I, and shear deflection, will introduce errors which tend to compensate each other. Especially is this true in the case of framed trusses, where the effect of shear distortion is relatively much greater. It has been found that the usual method of designing swing-bridge trusses, by the ordinary restricted formula, assuming I as constant and neglecting shear deflection, is justifiable, because the method of deflections, when applied to the resulting design, gives reactions which agree very closely with those found by the usual approximate method with the restricted form of the theorem of three moments. For plate-girder draw spans, however, the ordinary design formulas have been shown to introduce errors as large as from 10 to 12% on the side of danger, when the effects of variable moments of inertia are neglected.

* "Deflections and Statically Indeterminate Stresses," pp. 49–50.

† *Loc. cit.*, pp. 54-55.

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PAPERS AND DISCUSSIONS

THE INFILTRATION OF GROUND-WATER INTO SEWERS.

BY JOHN N. BROOKS, JUN. AM. SOC. C. E.

TO BE PRESENTED FEBRUARY 5TH, 1913.

An examination of the Index to the *Transactions* of the American Society of Civil Engineers, published recently, shows that the Society has never published a paper on the infiltration of ground-water into sewers. Some recently collected data on the subject are presented in Table 1, and a glance shows that the information is incomplete in most cases, and that there is a wide variation in the form in which it is presented.

The writer, therefore, has attempted to prepare a comprehensive and compact form for the presentation of data on infiltration, and to suggest rational units for the measurement of its quantity.

The factors governing the total quantity of infiltration into a sewer are:

1.—The diameter and length of the sewer.

2.—The material of which the sewer is constructed.

a.—In vitrified pipe sewers, the type of joint used.

b.—In concrete or brick sewers, the type and quantity of water-proofing used.

3.—The skill and care used in laying the sewer.

4.—The character of the materials traversed by the sewer.

5.—The relative positions of the sewer and the ground-water level.

NOTE.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

TABLE 1.—DATA ON INFILTRATION OF GROUND-WATER INTO SEWERS.

Item No.	Place.	Diameter of sewer, in inches.	Length of sewer, in miles.	Leakage, in percentage of capacity of sewer.	Leakage per day per mile of sewer, in gallons.	Leakage per day per capita, in gallons.	Total leakage, in gallons, per day.	Remarks.
1....	Boston, Mass..................	8 to 36	137	....	40 000	50	5 480 000	Before any connection to sewer.
2....	Massachusetts State..........	Various.	700	....	80 000	100	56 000 000	Densely populated section—800 people per mile of sewer.
3....	Kalamazoo, Mich............	....	....	20%	....	....	....	Before any connection to sewer.
4....	Norfolk, Va....................	....	....	60%	....	100	....	
5....	Schenectady, N. Y...........	....	....	5%	....	....	....	Wet ground and quicksand
6....	Canton, Ohio..................	....	11	....	26 500	*41	73 000	
7....	Taunton, Mass................	24	....	20%	....	....	....	Passing through swamp.
8....	North Brookfield, Mass.......	12	0.3	....	17 000	71	2 500–5 000	
9....	Rogers Park, Ill.....	6	1.7	....	1 240	*2	2 108	Deep sockets and careful ramming to minimize leakage.
10....	Brockton, Mass...............	....	16	....	25 000	31	400 000	
11....	Altoona, Pa.................	27 30 33½ by 44	1.2 0.6 0.95	....	40 814 86 592 264 000	*65 *145 *442	47 181 52 352 252 342	Brick and concrete sewer. Leakage reduced after careful watching of contractor.
12....	East Orange, N. J............	....	29	....	22 400	*43	650 000	10 ft. or more under water. Quicksand. Great precaution to prevent leakage.
13....	East Orange, N. J............	....	25	....	8 700	*11	217 500	
14....	Joint Trunk Sewer............	....	150	10%	25 000	25	3 750 000	Glacial drift and quicksand.
15....	Av. of Bloomfield, Orange, Montclair, and Glen Ridge	....	....	....	....	107	6 430 000	Average for the years 1895 to 1906.

* 600 people per mile of sewer.
Items 1 and 2. Report of State Board of Health. Discharge of Sewage into Boston Harbor. 1900.
" 3 to 12, inclusive. Report of F. P. Stearns, Past-President, Am. Soc. C. E., on the Sewerage of Mystic and Charles Rivers. Jan., 1899.
" 13 and 14. *Engineering Record*, Vol. 62, Oct. 1st, 1910, p. 377.
" 15. Passaic Valley Sewerage Commission, Report of Dec., 1907, p. 13.
" 14. Joint Trunk Sewer. Takes in sewers of part of Elizabeth, Roselle Park, part of Newark, West Orange, Summit, etc.

Reference to Table 1 shows that in every case data on some of these factors are missing. As it is evidently impossible to express all these factors in precise terms, it becomes necessary to agree on such terms, and the writer suggests the following:

For the material of which the sewer is constructed, in the case of pipe sewers, use the term "Vitrified Pipe." In the case of concrete sewers, state the thickness of the wall, the mixture, and whether crushed stone, gravel, or slag was used in the aggregate. For brick sewers, state the thickness of the wall, the quality of the brick, and the bond.

The type of joint is usually easily described. If the joints are of cement, it should be noted whether they are finished with a bevel or flush with the bell of the pipe.

For the type and quantity of water-proofing, if a fabric is used, the percentage of the total length of sewer treated may be stated, together with the number of plys of fabric and the kind of paint. In cases where a water-proofing compound is mixed with the concrete, a statement of the kind and quantity of compound will be sufficient.

For skill and care used in laying, use such terms as "Little," "Ordinary," and "Unusual."

For the character of the materials traversed, use such terms as "Wet," or "Dry," "Rock," "Gravel," "Sand," or "Clay."

For the relative positions of the sewer and the ground-water level, reference may be made to the daily reports of the engineer or inspector; and the length of wet trench reported may be expressed as the percentage of the total length of the trench, assuming that where wet trench is reported the sewer will be below the ground-water level.

The foregoing descriptive terms will inevitably have different shades of meaning for different engineers, but the writer believes that they are sufficiently definite to present the salient features of a sewer described by them with a degree of accuracy sufficient for an intelligent comparison with other sewers.

Reference to Table 1 shows that there is a wide variation in the units chosen for the measurement of the quantity of infiltration, and that none of them is altogether satisfactory. Expressing the quantity

of infiltration as a percentage of the maximum capacity of the sewer is irrational, because the maximum capacity depends in part on the gradient of the sewer, which has no relation whatever to the quantity of infiltration.

Gallons per day per mile of sewer is a useful unit only when the size of the pipe and the length of each size are stated, and in a collecting system with many laterals such a statement becomes decidedly cumbrous.

Gallons per day per capita of tributary population appears to be the favorite measure of the quantity of infiltration, but this unit is entirely irrational, for there is no conceivable relation between the number of people contributing sewage and the quantity of ground-water percolating into a sewer.

The total quantity of infiltration, in gallons per day, is a statement which is evidently entirely useless in making comparisons between systems unlike in diameter and length of conduit.

In view of the unsatisfactory form of the data, and of the units thus far considered, the writer suggests the presentation of such data in the manner shown in Table 2, and the following units for the measurement of the quantity of infiltration. These units are used by Mr. Nicholas S. Hill, Jr., and are believed by the writer to be both simple and rational.

For vitrified pipe sewers, the unit is gallons per day per foot of joint. Table 3 shows the total length of joints per 100 ft. of sewer for the commercial sizes of pipe, for which the total length of joints for any length of sewer is obtained by a single multiplication. Evidently, in a vitrified pipe sewer free from cracks, infiltration can occur only at the joints. An examination of Table 1 shows that sufficient data for the computation of infiltration, in gallons daily per foot of joint, are presented in only four cases, namely, Items 8 and 9, and in two sewers under Item 11. The data for these cases are repeated in Table 2, with the computed infiltration expressed in gallons per day per foot of joint; this is seen to vary from 0.3 to 5.0.

For concrete and brick sewers, the unit is gallons per day per square yard of interior surface. Table 4 shows the interior surface area, in square yards per 100 ft. of sewer, for circular sewers, from 3 ft. 8 in. to 6 ft. in diameter, and a single multiplication gives the interior surface area for any length of sewer. For sewers of egg-

TABLE 2.—DATA ON INFILTRATION OF GROUND-WATER INTO SEWERS.

Item No.	Sewer. Place.	Sewer. Type of section.	Sewer. Material.	Sewer. Diameter.	Sewer. Length, in feet.	Sewer. Skill and care in laying.	Sewer. Type of joints.	Sewer. Water-proofing; type and percentage of total length.	Wet trench; percentage of total length.	Character of material traversed.	Infiltration, in Gallons per Day. Per foot of joint.	Infiltration, in Gallons per Day. Per square yard of inside surface.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
8	North Brookfield, Mass.	....	....	12 in.	1 584	....	....	....	....	....	2.0	....
9	Rogers Park, Ill.	....	....	6 "	8 976	Unusual	Deep sockets.	....	....	....	0.3	....
11	Altoona, Pa.	....	....	27 "	6 336	....	....	....	....	....	2.6	....
	" "	....	....	30 "	3 168	....	....	....	....	....	5.0	....
16	....	Circular	Reinforced concrete, stone or gravel, 1 : 2 : 4.	3 ft. 8 in.	2 462	Ordinary	....	3-ply felt and pitch, 4 per cent.	80	Wet and dry gravel.	....	0.8
		"	"	4 " 3 "	7 220	"	....	"	"	"	....	
		"	"	4 " 4 "	1 904	"	....	"	"	"	....	
		"	"	4 " 6 "	11 066	"	....	"	"	"	....	
		"	"	4 " 9 "	5 838	"	....	"	"	"	....	
		"	"	5 " 0 "	3 574	"	....	"	"	"	....	
		"	"	5 " 3 "	2 901	"	....	"	"	"	....	
		"	"	6 " 0 "	12 779	"	....	"	"	"	....	

shaped or basket-handle cross-section it is necessary to compute the circumference for the special sections involved, but these computations will have been made in the design of the sewer long before the engineer is ready to make infiltration tests.

TABLE 3.—Vitrified Pipe.

Nominal diameter, in inches.	Laying length, in feet.	Length of each joint, in feet.	Length of joint per 100 lin. ft. of sewer, in feet.
(1)	(2)	(3)	(4)
4	2	1.05	52.5
6	2	1.57	78.5
8	2	2.09	104.5
10	2	2.62	131.0
12	2	3.14	157.0
15	2	3.93	196.5
18	2	4.71	235.5
21	2	5.50	275.0
24	2	6.28	314.0
27	2½	7.09	283.6
36	2½	9.42	376.8

TABLE 4.—Circular Sewers.

Diameter.	Inside area per 100 lin. ft., in square yards.
(1)	(2)
3 ft. 8 in.	127.99
4 " 0 "	139.63
4 " 3 "	148.35
4 " 6 "	157.08
4 " 9 "	165.81
5 " 0 "	174.53
5 " 3 "	183.26
5 " 6 "	191.99
5 " 9 "	200.71
6 " 0 "	209.44

Item 16 of Table 2 gives the results of a test for quantity of infiltration, recently made by the writer, under the direction of Mr. Hill, on a new reinforced concrete trunk sewer. Triangular weirs were set at two points, 47 745 ft. apart, and each weir was provided with an automatic head-recording device, giving a continuous record of change in head, and reading to the nearest $\frac{5}{1000}$ ft. From these records the average head on each weir for a period of one week was computed, by obtaining with the planimeter the area between the curve of change in head and an arbitrary base line,

and dividing by the proper length. From this average head the corresponding average rate of discharge was computed, making allowance for velocity of approach. The average time of flow between the weirs was obtained by the comparison of maxima and minima points on the respective curves showing decided change of head.

The interval or lag between the time of corresponding discharges having been determined as described, the total discharge at the up-stream weir for a period of one week was then subtracted from the total discharge at the down-stream weir during the corresponding period. The difference in discharge thus determined is a very close measure of the quantity of infiltration between the weirs during one week. The conditions for this test were ideal, as no sewage entered the trunk lines between the weirs during the test. The infiltration found was at the rate of 0.8 gal. per day per sq. yd. of interior surface.

A careful inspection of the length of sewer tested revealed no visible cracks, and the work in general appeared to be first-class, but the quantity of infiltration measured appears to be much smaller than may in general be expected in work of this class.

The writer hopes that discussion of this paper may bring out improvements in the form suggested for the presentation of data on the infiltration of ground-water into sewers, as well as additional data on the subject.

He wishes also to acknowledge his indebtedness to Mr. Nicholas S. Hill, Jr., for valuable criticism and suggestion in the preparation of this paper.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

A SUGGESTED IMPROVEMENT IN BUILDING WATER-BOUND MACADAM ROADS.

By J. L. Meem, Esq.

To be Presented February 19th, 1913.

In this era of vast road building throughout the country, it has seemed to the writer that there is too much variance in the specifications, especially in those for water-bound macadam roads, and this paper describes what is believed to be an improvement in such road construction.

By way of explanation, it may be stated that water-bound roads are those in which the only binder is water, and this is used very plentifully in the process of building the road. There is no other foundation than the rolled earth; the standard depth of loose rock ordinarily used is 9 in., and when this has been thoroughly rolled, a theoretical depth of 6 in. of finished roadway is obtained. The ordinarily accepted specification for placing this rock calls for 6 in. of No. 1 stone (the largest size) varying from $1\frac{1}{2}$ to 3 in. in diameter; then $2\frac{1}{2}$ in. of No. 2 stone, varying from $\frac{1}{2}$ to $1\frac{1}{2}$ in. in diameter, and the top course is finished off with stone $\frac{1}{2}$ in. and less in diameter, usually called "screenings," including the material resulting from the dust of fracture. Assuming, then, a depth of loose rock of 9 in. as a standard, the writer describes herein a method of proportioning this rock which he has used successfully and is using at the present time on an 8-mile section of the Memphis-Bristol Highway, in Tennessee.

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

All road builders, both engineers and contractors, will probably agree that the difficult and the essential thing to obtain in a water-bound road is the bond, and the method herein described attains this result at probably less cost than any other, and certainly at very much less cost than any the writer has ever tried.

The method consists in separating the screenings into two sizes, one size being from ½ to ¼ in. and the other the dust of fracture, and putting these two sizes on the road in separate operations, and the writer wishes to suggest the adoption of the following specifications for building a water-bound macadam road, both as to the method of proportioning the rock and as to sprinkling:

(*a*) The sub-grade shall be thoroughly rolled until it is firm and compact, with the proper crown of ¾ in. per ft.

(*b*) On this shall be placed 6 in. of No. 1 grade rock, ranging in size from 1½ to 3 in. This shall be spread uniformly and rolled dry until the rock does not creep before the roller, or creeps just enough to set it in place without crushing.

(*c*) Then 2 in. of stone of No. 2 grade, ranging in size from ½ to 1½ in., shall be spread uniformly, sprinkled, and rolled thoroughly into the voids of the No. 1 grade.

(*d*) Then ¾ in. of the coarse material from the screenings, consisting of sizes from ¼ to ½ in., shall be spread uniformly, sprinkled, and rolled thoroughly into the voids of the No. 2 grade.

(*e*) Finally, ¼ in. of the dust from the screenings shall be spread (preferably by hand from shovels), thoroughly soaked, and rolled to a finish.

This method gives a surface which is virtually impervious to water, and is less affected by automobile traffic than any with which the writer has had experience; in fact, it has been a matter of great surprise to him to see how well roads built by this method withstand automobile traffic without "raveling" or showing signs of deterioration.

From the contractor's point of view, as an economical method of building macadam roads, the writer finds that on the Memphis-Bristol Highway he is able to care for 100 cu. yd. of rock per day with one 10-ton roller, where previously, before the change was made, there was some difficulty in rolling properly one-half this quantity and getting the proper bond therein, in spite of the fact that it requires

one more operation to put on the extra course of stone. This is very much more than balanced by the quicker method of obtaining the required bond, and the final results as stated are so much more satisfactory than those obtained by using only three grades of rock and not separating the dust from the screenings that it would seem to be a material improvement in the method of road building, and should appeal to both engineers and contractors. It is only fair to say that the specification under which this section of the Memphis-Bristol Highway is being built called for the placing of the macadam in three operations, as originally stated, but that the change has been approved by the engineer in charge who now highly endorses the method.

This work requires a standard screen with a "dust jacket" (¼-in. perforations) over the ⅝-in. screen.

The writer, who is a member of the contracting firm which is building this section of the road, is interested both from the standpoint of the engineer and the contractor. He has never before seen this method advocated publicly, and invites discussion and criticism from other road builders.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

ON LONG-TIME TESTS OF PORTLAND CEMENT.

BY I. HIROI, M. AM. SOC. C. E.
TO BE PRESENTED FEBRUARY 19TH, 1913.

The behavior of Portland cement in sea water has long been a subject of discussion by chemists and engineers; yet the lack of complete experimental data on one hand and vast amounts of material and labor placed at stake in most harbor works of magnitude relying on the durability of cement, on the other, seem to make any results of investigation on its nature a matter of ever recurrent interest, especially to those engaged on maritime works. It is under such an impression that the writer presents some of the results of his still unfinished experiments, the completion of which, however, he may not live to see.

More than 15 years ago, at a time when the writer had to give out instructions for manufacturing concrete blocks for use in harbor works, he commenced a series of tests on the behavior of the cements then in use, in order to ascertain as far as possible the reliability of the rules laid down for the execution of the works. The tests have included the quantity and quality of water used in gauging, the quantity and quality of the sand and ballast, the differences of temperature, the kind of surroundings, the modes of fabrication, and, in short, most of the important points in the manufacture of mortar and concrete considered likely to affect the strength of finished work in course of time. Most of these tests have confirmed the correctness

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of the method as found in treatises relating to the subject and generally practiced in good works, so that it is hardly worth while describing here the results obtained. The durability test alone seems to be of some interest, particularly to those who have doubts as to the permanence of cement structures built in sea water.

The tests have been proposed to extend over 50 years, and although the time which has elapsed hardly covers one-third of that period, the results thus far obtained are of some value in showing the general trend of changes taking place in the strength of cement, and in forecasting in a certain measure the eventual results of the tests.

Three different kinds of cement and one hydraulic lime have been used for the various tests. In this paper only the results obtained with two kinds of cement will be considered. The chemical analyses of these two cements are given in Table 1.

TABLE 1.—Chemical Analyses of Cements.

	Cements.	
	A	*B*
Loss by combustion	0.88	1.24
Insoluble matter	1.30	1.25
Silica	22.20	19.00
Alumina	7.55	8.35
Ferric oxide	2.90	3.90
Lime	61.77	62.17
Magnesia	1.12	1.75
Miscellaneous	2.28	2.34

In quality, the cements satisfied the following requirements:

The cement shall be so fine that the residue on a sieve of 5 000 meshes per sq. in. will not exceed 10% of the whole.

The quantity of alumina contained in the cement shall not exceed 3 per cent.

The cement shall not commence to set within 1 hour of mixing with water.

The cement in setting shall not show any change in shape or volume.

Briquettes composed of 1 part of cement and 3 parts of standard sand, and immersed in sea water, shall have a tensile strength of not less than 130 lb. per sq. in. after 1 week, and gradually increasing to 170 lb. in 4 weeks, the amount of that increase to be not less than 30 lb. per sq. in.

All the briquettes for the durability test were made by hand, the mortar being gauged with fresh water and struck into the mould (with a trowel weighing 1¾ lb.) until water rose to the surface. The briquettes thus made were placed in a covered box for 24 hours, after which they were transferred to several media. The sands used for the mortar were of two kinds: (1) Coarse beach sand, with grains of all sizes up to about 0.06 in.; and (2) standard sand obtained by sifting the coarse sand between sieves with 400 and 1 400 meshes per sq. in. The briquettes kept in air were placed in an open shed, and those in water in tanks in which the water was changed once a week. The briquettes are uniformly 1 sq. in. in section, and five of them have been taken out for each single test.

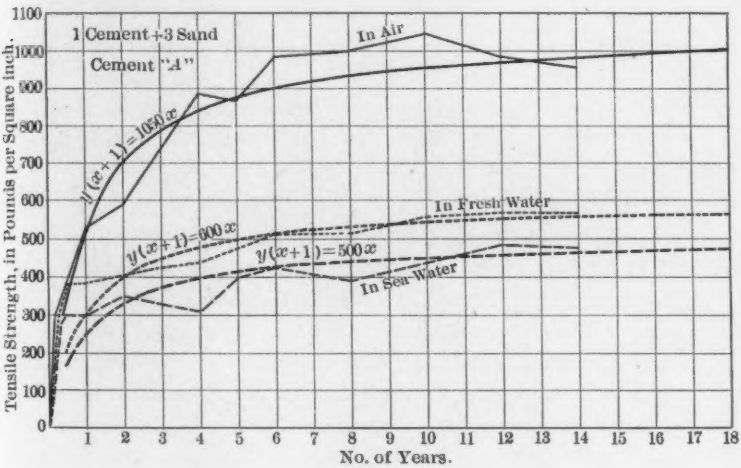

Fig. 1.

Fig. 1 shows the results of tests made with a mortar of 1 cement and 3 sand, by weight. These briquettes were placed separately in air, fresh water, and sea water, 24 hours after fabrication, in the manner already described, and remained there until tested. The strengths of briquettes made with coarse and standard sands have been found to be so nearly alike that the mean results were taken in plotting the curves. The strength attained by the use of finer sand naturally came out considerably lower. It will be seen, as might have been anticipated, that the mortar attains the greatest strength in air

and the least in sea water, the mean comparative ratio of strength attained being nearly as follows:

Air	1.00
Fresh water........................	0.56
Sea water..........................	0.45

The curves of strength thus traced are approximately hyperbolic, and may be more or less closely expressed by the following equations:

In air .... $y = \dfrac{1\,050\,x}{x+1}$

In fresh water .... $y = \dfrac{600\,x}{x+1}$

In sea water........... $y = \dfrac{500\,x}{x+1}$

in which $y =$ the tensile strength, in pounds per square inch, and $x =$ the number of years elapsed.

It will be seen that such mortars attain their greatest strength in about 6 to 10 years, beyond which, although they still continue (through occasional ups and downs) to increase in strength, the rate of increase is inconsiderable.

With neat cement briquettes (Fig. 2), the results are entirely different. Those kept in sea water attain their greatest strength in about 2 to 10 months, after which they rapidly decline, in some cases completely losing their strength in 4 or 5 years. Even then, however, not only are their forms intact, but they also show considerable crushing strength, which may amount to more than 6 000 lb. per sq. in. In the case of hydraulic lime placed in sea water, neat briquettes begun to show signs of disintegration from the outside, in about a year, and after about 4 years even the sound core, which had been decreasing in size all that time, finally succumbed, leaving a shapeless mass. In air and fresh water, neat briquettes continue to increase in strength for 4 or 5 years, when they attain much higher strength than those placed in sea water. A comparison of the air curves on Figs. 1 and 2 shows the decided superiority, in the long run, of the use of 1 cement + 3 sand mortar over neat cement. The total loss of tensile strength in neat cement briquettes, whatever may be the medium in which they are placed, appears to be a question of time.

Fig. 3 shows another series of tests made with briquettes which had been kept immersed in fresh water for 2 months before they were finally

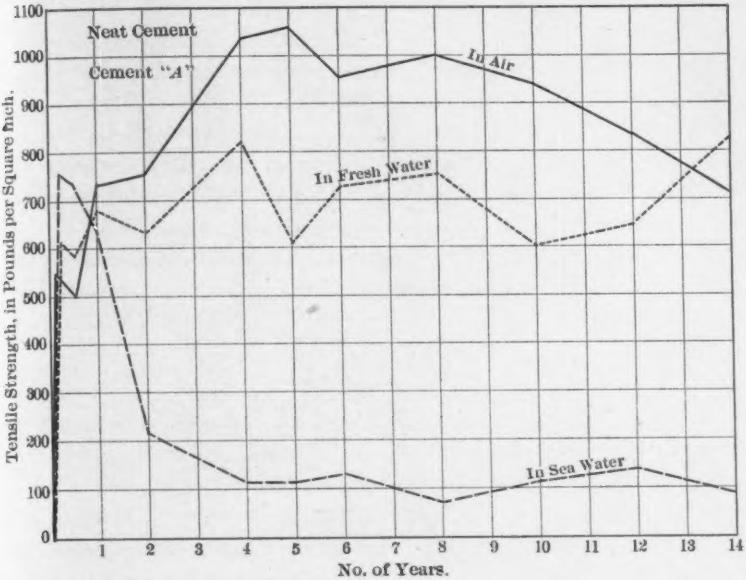
Neat Cement
Cement "A"
In Air
In Fresh Water
In Sea Water
Tensile Strength, in Pounds per Square Inch.
No. of Years.
0
100
200
300
400
500
600
700
800
900
1000
1100
1
2
3
4
5
6
7
8
9
10
11
12
13
14

FIG. 2.

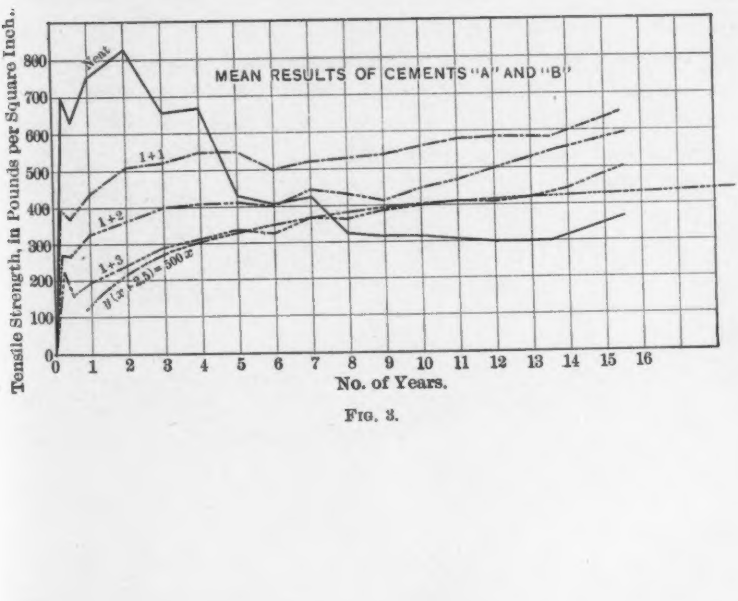
MEAN RESULTS OF CEMENTS "A" AND "B"
Neat
1+1
1+2
1+3
$y(x+2.5)=500x$
Tensile Strength, in Pounds per Square Inch.
No. of Years.
0
100
200
300
400
500
600
700
800
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16

FIG. 3.

placed in sea water in partial reproduction of the conditions which obtain with concrete blocks used in sea water. The briquettes are of neat cement, and of mortars of 1 cement + 1 sand, 1 cement + 2 sand, and 1 cement + 3 sand, all by weight. Each curve shows the mean of the results obtained for *A* and *B* cements. Although the general form of the curves, except that for the neat cement, are (as before) approximately hyperbolic, the increase of strength is not as rapid as in the case of briquettes immersed directly in sea water, shown in Fig. 1, but the rise is more steady and continues much longer. In the case of the 1 cement + 3 sand mortar, the regular curve given by the following equation approximately coincides with the actual one through the greater part of its length:

$$y = \frac{500\,x}{x + 2.5}.$$

As far as the comparison of this equation with the corresponding one in the preceding case bears out, the strength apparently attainable by such mortar appears to have the same limitation. With neat cement briquettes, the time in which the maximum strength is attained in this, compared with the previous case, is considerably retarded, taking place in about 2 years; and the rate of decline in strength is also much less rapid.

Although the results of these tests are not yet conclusive as to the permanence of strength of cements, their indications are that, with proper selection of materials, right proportions of ingredients, and with due attention to the mode of fabrication, cement mortars used under circumstances in which engineering structures are commonly built, will continue to increase in strength for indefinite lengths of time, and thus artificial stones made with them will be as enduring as natural ones of approved qualities.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

STREET SPRINKLING IN ST. PAUL, MINN.

Discussion.*

By C. L. Annan, M. Am. Soc. C. E.†

C. L. Annan, M. Am. Soc. C. E. (by letter).—In reply to Mr. Whinery, the following statement of unit costs is presented: Mr. Annan.

Total travel of 80 teams per day......	5 248 000 lin. ft.
Total effective travel (76%)..........	3 883 520 " "
Total area (width of spray, 22 ft.)....	9 493 050 sq. yd.

Daily expense:

80 teams @ $100 per month......................	$267
10 inspectors @ $75 per month..................	25
Stand-pipe repairs, average....................	15
Sprinkler " "	13
2 000 000 gal. water @ $90......................	180
Total	$500

Consequent cost of sprinkling 1 sq. yd. once. .0.00527 cent.
Average cost of sprinkling 1 sq. yd. four times per day for 210 days..................4.4 cents.

During the summer of 1910, 80 000 gal. of a 45% liquid asphalt, heated to 150° Fahr., were applied to 300 000 sq. yd. of macadam surface, the cost per square yard being as follows:

Cleaning street surface.....................	0.35 cent.
Road oil..................................	1.00 "
Application	0.15 "
Total	1.5 cents.

Three applications would be required for the full season, which would make the cost practically the same as for water sprinkling.

* Continued from November, 1912, *Proceedings*.
† Author's closure.

Mr. Annan.

There was much opposition and complaint of damage and annoyance by residents along the smeared streets, however, and, furthermore, as the oil dried out, it left the street surface in a badly disintegrated condition. During the following year a 20% oil was used. No heating was required; and three applications were sufficient for the season. The surface damage was less, but the popular outcry was not much abated.

In making the assessment, the factor of street widths is provided for in the trip element. Three trips are necessary to cover some wide streets; one trip suffices for a narrow street. The same frontage is assessed three times as much in one case as in the other, as a trip is a single line of travel in one direction.

The total cost of the season's sprinkling is prorated to the frontage participating. The law provides that 10% of this cost may be for equipment. The sprinkling of street intersections must also be regarded as a general charge, as there is no provision for payment otherwise. The law has also been construed to prohibit rebates for corner lots abutting on two sprinkled streets.

The matter of the concealed hydrant received attention early in 1910. Neither of the two eastern manufacturers approached could furnish just what was wanted, and it was late in the season before the desired article was obtained. In the meantime, the conclusion had been reached that the main objection to the stand-pipe is not its unsightly appearance. The ungainly sprinkler itself is no more esthetic, but worst of all is the faithful but inconsiderate horse at his 5 to 10-min. stands.

The teamster would view the innovation with no favorable eye. He would consider it a hardship to be forced to descend and climb again whenever he filled his tank, to say nothing of lifting the hydrant cover off and putting it on, and of stowing his loaded hose. The new hydrant was never put in.

The mud at the base of the stand-pipe, of course, must be eliminated as soon as possible by draining and paving, to insure the departure of a free tank.

When the St. Paul pavements are bare in freezing weather they are sprayed with calcium chloride (1 lb. per gal.) dissolved with a steam jet.

The following figures are approximately correct for a period of 3 weeks in December, 1910:

27 tons calcium chloride @ $14	$378
8 tons coal @ $4.75	38
4 teams, 18 days, @ $4	288
3 men, 18 days, @ $2	108
Total	$812

At intervals during the period, 100 000 sq. yd. of pavement were treated. Mr. Annan.

Mr. Blanchard asks: "If pavements are properly cleaned, * * * is there any necessity to sprinkle them to lay the dust? The answer is in the negative * * *."

Another answer is that the use of methods commonly considered effective does not down the demon "dust." Until perfection of method can be more nearly reached, a little water must be used here and elsewhere. Hand sweeping by day and power flushing by night and day do not hold dust in subjection. It is unceasingly making. It is tracked and blown in from adjoining unpaved streets, and arises from the sites of building operations. Whatever the theory, the condition must be met.

A light oil, especially designed for use on pavements, was tested last year on asphalt, brick, and stone surfaces in St. Paul, but with unsatisfactory results.

The vacuum cleaner may eventually do away with the use of the universal solvent, but it will be necessary to apply it to the sidewalks as well as the roadways, and even then will it get the impalpable, harassing dust emanating from all sources of friction in the busy life of a city's streets?

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

THE STRENGTH OF COLUMNS.

Discussion.*

By Messrs. J. O. Eckersley, Henry S. Prichard, and J. S. Branne.

J. O. Eckersley, M. Am. Soc. C. E. (by letter).—This brief discussion of Mr. Lilly's suggestive paper was prompted by the question: Would it not repay the endeavor to derive a more rigorous treatment for the strength of columns? Science is truly a function of the epoch in its ideals and achievements, and technical knowledge, as we understand it to-day, is becoming pre-eminently mathematical, and justly so, for mathematics unchallenged governs the appreciation and mastery of the physical sciences in theory and practice. Mr. Eckersley.

Even though one has the feeling that the solution, in an absolute sense, founded on abstract mathematical reasoning, will be found impossible; at the same time, one recognizes the importance of the endeavor to attain precisely this result.

Although the demands made on mathematical science, to follow Mr. Lilly's paper, are not great, yet, if the rough and ready reasoning of the engineer gave way to the more rigorous analysis, a considerable knowledge in the realm of higher mathematics would be required.

In general, engineers, almost unanimously, regard analyses involving higher mathematics as dispensable. Judging from past experience, however, there appears to be every inducement for the mathematical engineer to continue his studies with unflagging zeal and confident optimism, for there is little reason to doubt, and many reasons to expect, greater achievements through just such persistent pursuit of scientific investigation.

This attitude of engineers toward analyses is clearly shown in their endeavor to simplify everything, an endeavor economically and entirely commendable, which, however, introduces simplifying assump-

* Continued from October, 1912, *Proceedings*.

Mr. Eckersley.

tions such that it is questionable whether we may with propriety consider them valid, at least from a mathematical viewpoint.

The syncopation, we might say, of the column formula is no exception. For instance, Rankine's, written in the form, $y = a\ (1 + bx^2)^{-1}$, is a cubic equation; this was simplified to a quadratic by substituting a parabola in the form of $y = c - dx^2$; and, in due time, it was further reduced to the linear form, $y = e - fx$, where the letters, a to f, inclusive, represent empirical constants determined in every case as if the simplifying assumptions of the theory of flexure, initial straightness, homogeneity, central loading, etc., were rigorously obtained. The questioned validity of the deductions from such formulas is shown by the fact that the results of experiment and the deductions are not confirmatory. More exact mathematical treatment of the question than that given by the above formulas has been advanced by several writers, English, French, German, and American; also, the effects of eccentricity and local bending between points of attachment of lattice bars are not new. Mr. Lilly's treatment of the subject, in common with others, has introduced approximations at divers places in the analysis, which permit of criticism from the mathematical standpoint.

To compare some of the salient points of difference between what we may term the rigorous, as compared with the usual, approximate treatment, let us consider the following parallel columns for reference:

	Rigorous.		Usual Approximation.
1st.	When a column is subjected to strain, its dimensions in every direction are altered, the lateral changes following a law similar to that of Hooke.	1st.	Changes of dimensions are construed as negligible.
2d.	Strains do occur in all the three mutually perpendicular planes of reference.	2d.	The strains are assumed to be entirely co-planar, *i. e.*, in one plane.
3d.	A torque would, in general, exist.	3d.	Irrotational stress is assumed.
4th.	The elastic curve, when the column weight is considered, is not a symmetrical curve.	4th.	The weight of the column is neglected, which leads to a symmetrical elastic curve.
5th.	With suitable lateral forces, the so-called neutral plane may be broken or even discontinuous.	5th.	So far as the geometry of the neutral line is concerned, it is regarded as a continuous curved line having a tangent and osculating plane.

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Rigorous.	Usual Approximation.
6th. The curvature would, in general, be tortuous.	6th. In the curvature formula, it is assumed that $dl = dx$, which reduces the exact expression, $\frac{dx d^2 y}{dl^3}$, to $\frac{d^2 y}{dx^2}$. (Where the curve is such that a great difference in x occurs for a small difference in y, the exact form should be retained.)
7th. The section after strain is actually curved.	7th. The section is assumed to remain plane.
8th. Vertical shear-deformation exists.	8th. Vertical shear-deformation is neglected.
9th. The conjugate effect of transverse strain really transforms the surface of a rectangular sectioned column into an anticlastic surface.	9th. Conjugate effect of transverse strain is neglected.

To examine the column by the more exact treatment, recourse must be had to the general equations of the equilibrium of an elastic solid, and would lead to expressions involving determinants, elliptic functions, Bessel equations, and what not, and is beyond the scope of this discussion. The column would be introduced by definition, endowed with certain ideal properties, and subjected to certain conditions not confined to empiric verification, just as a set of necessary and sufficient axioms is adopted for the various geometries, and the translation, into the language of the applied mathematical treatment, would be accomplished by substituting as many of the properties of the materials as could be experimentally determined or inferred, in lieu of the ideal.

The essential difference between such an analysis and the usual method is that evanescent quantities of a higher order would be retained and their final elimination made when the derivative governing the desired degree of accuracy was determined; while the usual method neglects certain factors in the initial assumptions and later approximations, believing that their effect on the final result is negligible, which, of course, is admissible when the degree of approximation permits, that is, when the displacements are expressed as functions of some selected system of spatial co-ordinates and are developed into series—the approximation ends with the first derivative, and we have the usual Hooke's assumption, of a plane remaining plane under strain. This order of accuracy, however, is only an approximation; yet, under certain conditions, it is sufficiently close.

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What the writer wishes to bring out clearly is this: A vast literature has developed on the subject, both at home and abroad, and nowhere has he found what could be called a rigorous solution from the purely "mathematical theory of elasticity" point of view. The question in mind while writing this is: Would it not be profitable to attack the problem from such a point of view, and then use this as a means toward the interpretation and co-ordination of the phenomena? What if the analysis does involve the theory of probabilities, determinants, and higher functions? It is the pursuit along these lines which offers any reasonable excuse to engineers for the existence of anything other than a straight-line formula; for the problem from an engineer's point of view is simply this: Primarily, for very short prisms (so short that no lateral forces act), the crushing strength represents the value of the function. Secondarily, for very long prisms, in general (even though lateral forces act), the bending value is the controlling factor, and represents the function. Hence, all that the engineer desires is a rational formula giving a continuous relation between these two limiting values, even though it contains approximations.

It is obvious how complex the relations between the crushing, shearing, and bending values of the material would be for intermediate lengths, aside from the influence of local bending due to lattice bars, etc. The true form of the expression involving these would be of little interest save to the mathematical engineer, for it must be conceded that, from a practical point of view, a straight-line formula can be used successfully in the design of columns.

Finally, Mr. Lilly is to be congratulated and encouraged to continue his investigations, which will, it is believed, approach more nearly the more rigorous treatment as he considers the neglected factors.

Mr. Prichard.

HENRY S. PRICHARD, M. AM. SOC. C. E. (by letter).—The author considers the Rankine-Gordon formula to be the best of the various formulas which have been put forward to determine the strength of columns, and in support of his opinion he quotes the statement that: "The theoretic basis of Rankine's formula seems far more satisfactory than that of any other which has been proposed"; but he qualifies his opinion in respect to "theoretic basis," by stating that an assumption is made in the derivation of the formula which "errs on the side of safety."

In dealing with the practical side of his subject, the author gives, in Fig. 4, a curve, *A*, showing the results of some tests which he made on mild-steel solid columns, ½ in. in diameter, and another curve showing the results obtained by using Rankine's formula with constants, which he, the author, has deduced for mild steel; and he states that he "has shown that the experimental curves can be closely approximated

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by assuming the deflection curve of the column to vary in some proportion less than the square of the length"; that "the resulting formulas then become more complex and are of little use for the practical design of columns," and that "for this reason" he "adheres to the Rankine-Gordon as a practical working formula."

In this comparison of the Rankine-Gordon formula with the results of experiments, the constant used as "the strength to compression" is 80 000 lb. per sq. in. The writer does not share the author's opinions as to the theoretical and practical excellence of the Rankine formula, and he does not approve of the use of such a high unit stress for "the strength to compression."

Rankine, before giving his analysis of beams and columns, states that "the elasticity of every solid is sensibly perfect when the strain does not exceed a certain limit," that "Hooke's Law—*'ut tensio sic vis'*—is sensibly true for all relations between strains and stresses," and that "this condition is fulfilled in nearly all cases in which the stresses are within the limit of proof strength"; and he bases his analyses on these fundamental statements; in other words, he predicates ideal conditions.

In his analysis of beams, he derives equations for deflections based on the fact—quite true for beams—that the entire stress in the extreme fiber is due to bending. Subsequently, he makes the mistake of applying one of these equations to the deflection of columns, entirely overlooking or neglecting the fact that, in all cases of ordinary columns, part, at least, of the stress in the extreme fiber is due to direct compression. This mistake leads to an erroneous formula for columns, which, on the strength of Rankine's endorsement, has deceived and has continued to deceive engineers, and interfere with a correct understanding of the subject, for more than fifty years.

The error is reflected in the author's Equation 6 for deflection of columns, which he makes a factor of the total stress intensity in the extreme fiber f, instead of a factor of $f - p$, as it should be (p being the load per unit area). The author prefaces this substitution with an unwarranted assumption, which, as he states, "errs," and gives what he terms "the first approximation." When f has a value of 80 000 lb. per sq. in. and the length is 60.8 times the radius of gyration, the effect of this substitution is to change the result of his Equation 7 from 80 000 to 40 000 lb. per sq. in.

It is astonishing that Rankine should have made, or acquiesced in, this error in regard to the deflection of columns, as a centrally loaded column, made of perfectly elastic material and having an initially straight axis, is truly a spring, and he has proved the following:*

* "Applied Mechanics," Eleventh edition, pp. 351–352.

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"THEOREM. *That a spring of a given length and section, to the ends of whose neutral surface a pair of forces are applied, will not be bent if those forces are less than a certain finite magnitude.*"

He develops the equation, $P = \dfrac{\pi^2 E I}{l^2}$ (which is Euler's), and adds, "and this *finite quantity* is the *smallest force which will bend the given spring* in the manner proposed." For instance, a centrally loaded, perfectly elastic, initially straight spring, with a modulus of elasticity of 30 000 000 and a ratio of length to radius of gyration of 60.8, will not be bent if the load is less than 80 000 lb. per sq. in.

There is another limit to the value of P; it cannot, of course, be greater than $f A$ (limiting stress intensity times cross-sectional area), and this latter limit governs for short columns, as the force required to strain such columns to the limiting stress intensity is less than that which it would take to bend them, if they were not thus limited. For instance, if a steel spring be conceived as having a ratio of length to radius of gyration of less than 60.8, and if the limiting stress intensity is 80 000 lb. per sq. in., the stress will reach this limit under a load of 80 000 lb. per sq. in. without any preliminary bending.

In view of the prevalent uncertainty as to the theory of perfectly elastic, initially straight, centrally loaded columns, the writer suggests that the author state whether he accepts this theorem of Rankine's; and if he does not accept it, that he indicate wherein he considers Rankine's "proof" to be in error.

For the limiting stress intensity given by the author (80 000 lb. per sq. in.), the limiting line, on Fig. 4 (beyond which initially straight, perfectly elastic, mild-steel columns will either crush or become plastic and flow, if short, or begin to bend and almost immediately fail,* if long), will follow the 80 000 lb. per sq. in. line from $\dfrac{l}{\rho} = 0$ to its intersection with Euler's curve at $\dfrac{l}{\rho} = 60.8$, and from this point it will follow Euler's curve for all longer columns. The difference between this theoretically correct line and the line, Curve *2*, of the author's adaptation of Rankine's formula, shows the latter to be in error as a theoretical formula, for such ideal steel under such ideal conditions, from 0 to 50 per cent.

Of course, steel which crushes at 80 000 lb. per sq. in. is not perfectly elastic to this limit, and, of course, the other ideal assump-

* In a paper on the "Theory of the Ideal Column," in *Transactions*, Am. Soc. C. E., Vol. XXXIX (1898), pp. 100–102, William Cain, M. Am. Soc. C. E., has shown: "first, that Euler's formula gives the load at which bending just begins, and, second, that a very small increase to this load insures failure." As a numerical illustration, a column pivoted at the ends, 325 in. long, was assumed as built up of two 5-in. channels. The inch being the unit, $A = 3.9$, $I = 14.8$, and $E = 29\,000\,000$ lb. per sq. in. Euler's formula gives the load that causes incipient bending as 40 105 lb. It was computed by an exact formula that an increase of the load of only 5 lb. caused a deflection of 3.44 in. at the center, with a resultant stress on the most compressed fiber greater than the elastic limit. Finally, an increase of 2 or 3 lb. more would entail rupture, or a breaking in two of the column.

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tions of theory are not fully realized. Actually, columns are never quite initially straight and centrally loaded. There is always some initial deviation between the axis and the line of thrust (though it is not always appreciable), and this causes some deflection under any load, though in many cases it is very slight and in some it is not appreciable until the load is nearly equal to either Euler's limit, if the column is long, or to the yield point, if it is of moderate length.* Actually, the limiting stress at which failure occurs, in wrought-iron and steel columns which are too short to have Euler's formula apply, is not constant (even for material of the same quality), as indicated by theory, but varies from a stress somewhat below the yield point, for the longest columns of this class, to the yield point, for short columns, and to the crushing stress for very short ones. This is due to the great reduction in stiffness which takes place at the yield point and to the partial reduction in stiffness from imperfections in elasticity, which develops shortly before the yield point is reached.

The loss in stiffness necessary to cause failure by bending grows less as length increases; hence, the load required to cause the failure of columns of this class grows less as length increases, even when great care is taken to reduce to a minimum the initial deviation between the axis and the line of thrust.

The fact that the strength of wrought-iron and soft or medium steel columns of moderate length is limited to about the yield point, is well illustrated in Tables 2 and 3, and many other series of column tests could be cited if necessary. In fact, the writer does not know of any such columns of moderate length which sustained loads greater than the yield point of the material. In experiments, short columns, solid or of thick material, do, temporarily, at least, sustain loads in excess of the yield point, and their ability to do so has a value as a temporary safeguard against disaster under excessive loads, even though the columns would be irreparably injured. This justifies a somewhat higher working load for such columns than would otherwise be the case.

Short wrought-iron and steel columns of such solid or thick material, however, are not much used. Most short columns are made of such thin material, have their cross-sections so distended, or have some detail so weak, that if tested to destruction, failure would occur

* In illustration the following tests are cited:

A 4-in., wrought-iron, I-beam column, 6 ft. 6¼ in. long with rounded ends and a ratio of length to radius of gyration of 161, tested by Mr. Christie, at Pencoyd, had no appreciable deflection until the load almost reached 10 300 lb. per sq. in., at which point it deflected 0.03 in. It failed at 12 113 lb. per sq. in. Euler's formula for this ratio of length to radius of gyration and for a modulus of elasticity of 27 000 000 gives 10 300 lb. per sq. in. (*Transactions*, Am. Soc. C. E., Vol. XIII (1884), p. 111.)

The Watertown Arsenal Report for 1910, p. 172, describes a steel column, 13 ft. 8¾ in. long, composed of one web 10 by ⅜ in., and four angles, 4 by 3 by ⅜ in., tested with flat ends, which had no deflection up to and including 32 000 lb. per sq. in. At 34 000 lb. per sq. in., the deflection was 0.17 in. and at 34 710 lb. per sq. in., it failed by deflection. The ratio of length to radius of gyration was 100. The yield point, as shown by tests of three similar columns with a ratio of 12, was 36 000 for this grade of steel.

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TABLE 2.—Yield Point in Tension and Compression, and Failure in Compression.

All bars from the same blow of Bessemer steel.

All specimens of each size from the same bar, and all of full size, "as from the rolls," except the tension tests of 3-in. flats.

All compression specimens tested with flat ends.

The specimens used for determining the yield points in compression were two diameters long.

All results are given in pounds per square inch.

Compiled from tests made by the late Charles A. Marshall, M. Am. Soc. C. E., and given in *Transactions*, Am. Soc. C. E., Vol. XVII (1887), p. 68, Tables 1 and 2.

Shape of bar and diameter, in inches.	Yield Point.				Failure Loads in Compression. Ratios of Length to Radius of Gyration are Given in Columns Headed $\frac{l}{\rho}$.							
	Tension.		Compression.		1 Test each.		1 Test each.		1 Test each.		1 Test each	
	No. of tests averaged.	Load.	No. of tests averaged.	Load.	$\frac{l}{\rho}$	Load.	$\frac{l}{\rho}$	Load.	$\frac{l}{\rho}$	Load.	$\frac{l}{\rho}$	Load.
¾ Round.	4	46 090	1	47 300	50	44 980	..	......	..	......	..	......
1 "	4	44 202	2	46 020	48	44 000	..	......	..	......	..	......
1¼ "	3	40 747	2	43 460	48	40 850	..	......	..	......	..	......
1½ "	3	40 275	2	41 290	48	41 880	..	......	..	......	..	......
1¾ "	3	40 017	2	42 075	48	39 950	..	......	..	......	..	......
2 "	3	38 207	2	38 830	48	40 850	..	......	..	......	..	......
2¼ "	1	37 000	2	38 125	48	36 790	72	36 580	..	......	..	......
2½ "	1	36 100	2	36 840	48	35 650	70	34 450	..	......	..	......
¾ Square.	3	44 273	2	43 845	45	44 960	..	......	..	......	..	......
⅞ "	2	47 815	2	49 055	..	......	..	......	..	......	..	......
1 "	3	43 560	1	44 025	42	43 080	..	......	..	......	..	......
1¼ "	3	41 060	2	42 300	42	40 060	..	......	..	......	..	......
1½ "	3	39 317	2	42 740	42	38 420	..	......	..	......	..	......
1¾ "	3	38 193	2	40 630	42	39 450	..	......	..	......	..	......
2 "	No	test	1	39 870	42	39 750	..	......	..	......	..	......
2¼ "	1	38 310	2	39 940	42	39 270	63	37 330	..	......	..	......
3×⅜ Flat.	3	47 363	..	......	42	47 650	63	46 420	83	46 150	..	......
3×½ "	3	44 417	..	......	42	41 650	..	......	83	43 490	104	42 170
3×¾ "	3	41 447	..	......	42	43 580	..	......	..	......	102	40 700
3×1 "	3	39 397	..	......	42	41 200	..	......	83	36 920	..	......
3×1¼ "	4	38 482	..	......	39	38 350	63	35 430	..	......	..	......
3×1½ "	4	37 820	..	......	42	36 920	..	......	..	......	..	......
3×1¾ "	4	35 917	..	......	42	35 760	..	......	..	......	..	......
3×2 "	4	39 302	..	......	42	37 670	..	......	..	......	..	......
4×½ " *	2	53 800	..	......	42	55 420	63	55 110	83	55 320	104	51 210
4×1 "	2	41 415	..	......	42	42 630	..	......	..	......	104	39 270
4×1¼ "	1	36 680	..	......	42	39 300	..	......	83	38 200	..	......
4×1½ "	1	37 580	..	......	42	39 500	63	39 850	..	......	104	39 100

* "Bar finished at very low heat."

Note that the failures by compression occurred at or below the yield point, that the yield point of each bar was about the same in tension and compression, and that it varied greatly in different sizes.

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TABLE 3.—COMPRESSION TESTS, AT WATERTOWN ARSENAL, OF ROLLED STEEL H-SECTIONS. NOMINAL SIZE, 6 BY 6 IN., 23.8 LB. PER FT., TESTED IN HORIZONTAL POSITION, WITH WEBS VERTICAL.

The "elastic limit" per square inch in tensile tests of four specimens from flange and two from web is given as follows: From flange, 28 500, 31 840, 29 500, 30 490; average, 30 107. From web, 32 590, 31 850; average, 32 245.

Compiled from Watertown Arsenal Reports for 1908 and 1909.

Ratio of length to least radius of gyration.	TESTED WITH FLAT ENDS.			TESTED WITH PIN ENDS. PINS VERTICAL, IN PLANE OF WEB, AND 3 IN. IN DIAMETER.		
	Yield point, in pounds per square inch.	Ultimate, in pounds per square inch.	Initial horizontal deflection.	Yield point, in pounds per square inch.	Ultimate, in pounds per square inch.	Initial horizontal deflection.
25	30 000	43 110	None given	31 000	45 000	None given.
25	30 000	40 716	" "	29 000	44 000	" "
25	30 000	44 750	" "	29 000	46 000	" "
50	No yield point before ultimate was reached.	28 980	" "	....	28 850	0.08, 0.11 in.
50		28 620	" "	29 000	33 350	Kinky + 0.04 in.
50		29 000	" "		30 780	0.04, 0.03 in.
75		29 000	" "	No yield point before ultimate was reached.	28 000	0.07, 0.4 in.
75		29 000	0.04 in.		28 000	0.20, 0.12 in.
75		28 630	0.04 in.		28 650	0.07, 0.00 in.
100		28 000	wavy		24 000	0.23, 0.18 in.
100		26 000	0.00 in.		26 000	0.15 in.
100		28 000	0.00 in.		27 000	0.07, 0.10 in.
125		27 000	0.08 in.		22 540	0.10 in.
125		26 000	0.10 in.		23 925	0.20, 0.15 in.
125		25 000	0.17 in.		25 000	0.05, 0.06 in.
150		24 670	0.03 in.		20 360	0.15 in.
150		23 000	0.00 in.		17 430	0.16 in.
150		23 000	0.17 in.		10 100	0.42 in.

Note the great importance of the yield point in limiting ultimate capacity.

from local bending (or, as the author aptly terms it, secondary flexure), or from the giving way of some detail, under a load not much, if any, greater than the yield point.

The yield points of wrought iron and steel are, beyond question, the critical stresses for wrought-iron and steel columns, and should be substituted for the crushing resistances given in Table 1 as "the strength to compression."

In this connection, it is useful to know that the yield point of steel (and probably of wrought iron) is practically the same, for metal of the same quality, in compression and tension. This is illustrated in Tables 2 and 4 (after allowing for the fact that the yield points in compression, in Table 2, were determined from very short specimens, which tends, whether the stress be compression or tension, to raise somewhat the yield point), and many other cases could be cited if necessary.

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TABLE 4.—COMPARATIVE TESTS IN TENSION AND COMPRESSION OF SPECIMENS, $1\frac{1}{4}$ IN. IN DIAMETER, FROM OPEN-HEARTH, STEEL BARS OF TEN GRADES.

Specimens turned to 1.0092 in. in diameter. Compression specimens 12 in. long, with ratio of length to radius of gyration of 48.
Compiled from Watertown Arsenal Reports for 1886 and 1887.
All loads in pounds per square inch.

Mark.	COMPOSITION.			TENSION.		COMPRESSION.	
	C., per cent.	Mn., per cent.	Si., per cent.	Ultimate.	Yield point.	Yield point.	Ultimate.
833	0.09	0.11	....	52 475	30 000	30 500	32 125
123	0.20	0.45	....	68 375	39 500	37 000	39 190
782	0.31	0.57	....	80 600	46 500	44 500	45 500
795	0.37	0.70	....	85 100	50 000	47 000	50 875
803	0.51	0.58	0.02	98 700	58 000	57 000	58 000
797	0.57	0.93	0.07	117 400	56 000	57 000	65 500
823	0.71	0.58	0.08	116 750	56 000	56 000	65 440
750	0.81	0.56	0.17	149 600	73 000	76 500	87 750
756	0.89	0.57	0.19	141 290	76 000	77 500	84 125
334	0.97	0.80	0.28	152 550	80 000	83 500	91 500

Note that the yield point in tension and compression is about the same for each grade, and that failure of compression specimens of the five lowest grades occurred near the yield point.

Digressing, somewhat, from the direct theme of this discussion: Some engineers, in discussing the question of local slenderness or weakness, have advocated the doctrine that columns should be designed so that they cannot fail from local slenderness or weakness, but will, if tested, develop the full strength of the column considered as a whole. This is entirely a matter of economy and convenience, and of having ample stiffness and sufficient strength to carry the load required with an ample margin of safety. If the column design is good in these respects, there is no reason whatever for changing to a less economical or convenient form of cross-section. The cross-section usually has to be designed with reference to connections, method of loading, and other practical considerations, and, when thus designed, is likely to have section and dimensions which, though amply strong and stiff, will not develop the strength of the column, as gauged by its length, radius of gyration, and cross-sectional area, unless sectional area not needed for any other purpose is added. The important consideration is that the strength of a column shall be gauged by its weakest feature.

Reverting to a direct discussion of the paper: The Rankine formula is not a satisfactory one for practical use. It cannot be made to agree well with the results of tests of wrought-iron and steel columns of moderate length, and, when the constants are adjusted to make the formula agree as well as may be with the results of tests of such columns, it gives results much too high for long columns. On the

other hand, if, by suitable constants, the formula is made to give proper values for long columns, it gives results unnecessarily low for those of moderate length, and, with the same constants, may give results altogether too high for short columns, as is the case with the author's adaptation of the formula, as gauged by his own experiments. These facts are shown in Table 5, in which it is also shown that, when an allowance is made for slight unintentional eccentricity, to the extent recommended by A. Marston,* M. Am. Soc. C. E., the author's recent experiments confirm with remarkable closeness the theory of eccentrically loaded columns. The comparison is all the more striking and convincing from the fact that the experiments were made by an advocate, on theoretical grounds, of the Rankine formula, who is, presumably, free from any prejudice in favor of the theory which, with some variation in detail, is supported by the analyses of Messrs. Marston, Cain, Moncrieff, Johnson, Jonson, and others who have discussed the subject of columns before this Society. Mr Prichard.

Marston's equation for nominally centrally loaded columns, and the close approximations to Marston's equation which others have derived, are tedious to apply, but they can be closely approximated as to results by easily applied empirical formulas. The following empirical formulas agree well with the results of the author's experiments on solid mild-steel columns, ½ in. in diameter, except in the case of very short ones, which failed at loads above the yield point; (these loads were, doubtless, much higher than those which the columns could permanently sustain without undue deformation).

For ratios of length, l, to radius of gyration, ρ, of 100 or less,

$$p = 58\,000 - 3\left(\frac{l}{\rho}\right)^2 \quad\ldots\ldots\ldots\ldots\ldots\ldots (a)$$

For ratios of length to radius of gyration of 100 or more,

$$p = \frac{300\,000\,000}{\left(\frac{l}{\rho}\right)^2 + 7\frac{l}{\rho}} \quad\ldots\ldots\ldots\ldots\ldots\ldots (b)$$

p, being the load per square inch of cross-sectional area.

These equations are arranged for the sole purpose of formulating the author's experiments on solid mild-steel columns, ½ in. in diameter, and, as arranged, are presented without recommendation for any other purpose. When made entirely general for any grade of steel or wrought iron, they become

$$p = \text{yield point} - \text{constant}\left(\frac{l}{\rho}\right)^2 \quad\ldots\ldots\ldots\ldots (c)$$

$$p = \frac{\pi^2 \times \text{modulus of elasticity}}{\left(\frac{l}{\rho}\right)^2 + x\left(\frac{l}{\rho}\right)} \quad\ldots\ldots\ldots\ldots (d)$$

* *Transactions*, Am. Soc. C. E., Vol. XXXIX (1898), pp. 108-113.

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TABLE 5.—COMPARISON OF LOADS CAUSING FAILURE OF SOLID MILD-STEEL ROUND-ENDED COLUMNS, $\frac{1}{2}$ IN. IN DIAMETER, TESTED BY W. E. LILLY; AS SCALED FROM FIG. 4, WITH RESISTANCE INDICATED BY THEORY, RANKINE'S FORMULA, AND EMPIRICAL FORMULAS, a AND b (Equations a and b).

All results are given in pounds per square inch.

Ratio of length, l, to radius of gyration, ρ.	THEORY (MARSTON*). $p = \dfrac{58\,000}{1 + \dfrac{e\,y}{\rho^2} \sec.\left(\dfrac{1}{2}\dfrac{l}{\rho}\sqrt{\dfrac{p}{E}}\right)}$ y = distance from axis to extreme fiber. e = unintentional eccentricity.		Results of W. E. Lilly's experiments; as scaled from Curve A of Fig. 4.	RANKINE'S FORMULA. As given by W. E. Lilly. $p = \dfrac{80\,000}{1 + \dfrac{\left(\frac{l}{\rho}\right)^2}{\dfrac{\pi^2 E}{80\,000}}}$	RANKINE'S FORMULA. As arranged for moderate lengths. $p = \dfrac{62\,000}{1 + \dfrac{\left(\frac{l}{\rho}\right)^2}{10\,000}}$	Equation a $p = 58\,000 - 3\left(\frac{l}{\rho}\right)^2$ Equation b $p = \dfrac{300\,000\,000}{\left(\frac{l}{\rho}\right) + 7\frac{l}{\rho}}$
	$e = 0.0$	$e = 0.00375$ in.				
						a
20	58 000	54 400	61 500	72 200	59 600	56 800
40	58 000	53 200	53 500	55 800	53 400	53 200
45	58 000	52 600	52 000	51 700	51 600	51 900
60	58 000	49 400	49 500	40 500	45 600	47 200
70	58 000	45 300	46 000	34 400	41 600	43 300
80	46 200	39 300	41 500	29 300	37 800	40 800
90	36 500	33 000	34 500	25 300	34 300	33 700
100	29 600	27 600	28 000	21 600	31 000	28 000
						b
120	20 600	19 800	19 700	16 350	25 400	19 700
140	15 100	14 700	14 500	12 700	20 900	14 500
160	11 600	11 400	11 200	10 100	17 400	11 200
200	7 400	7 320	......	6 770	12 400	7 200
300	3 290	3 270	......	3 160	6 200	3 260
400	1 850	1 846	......	1 810	3 650	1 840

*First published by A. Marston, M. Am. Soc. C. E., in *Transactions*, Am. Soc. C. E., Vol. XXXIX (1898), pp. 108-113, and since endorsed by many authorities. Marston used a value of 0.06 for $\frac{e\,y}{\rho^2}$ in comparing his formula with Tetmajer's tests of mild steel. This value applied to $\frac{1}{2}$-in. round bars gives $e = 0.00375$ in., as above.

Note that Rankine's formula cannot be made to agree with the "Experiments," while "Theory" agrees very closely when slight eccentricity is assumed.

Mr. Prichard.

The range of application of Equation *c*, and the value of its constant, have to be chosen to suit the series of columns for which the formulas are arranged. The value of x has then to be found by first determining the value of p for the length of column at which the application of Equation *c* ends and Equation *d* begins.

Equation *c* is a parabola; a form which the late J. B. Johnson, M. Am. Soc. C. E., advocated for short columns and those of moderate length. He, however, made his parabolas always tangent to Euler's curve, and advocated Euler's formula for long columns. On this basis, the constant for columns with pivoted ends is always

$$\frac{(\text{yield point})^2}{4\,\pi^2 \times \text{modulus of elasticity}}.$$

The writer prefers to increase the constant and modify Euler's formula for long columns, as per Equation *d*, in order to make a greater reduction for length. For bridges and buildings, long columns are usually tabooed, but for some kinds of construction they are not objectionable.

Professor Johnson compared his column formula with the very careful tests made by M. Considère. The tests with which the formulas were compared were of columns consisting of rectangular steel bars with pivoted ends, adjusted so that there was no appreciable eccentricity. The bars were of six degrees of hardness, and the results when plotted agreed with the corresponding six parabolic curves of the formula to an extent that was truly remarkable.

To show that the strength of a column, unless it is very short, is no function of the ultimate strength of the material, either in tension or compression, M. Considère cold-rolled the medium-hard steel, No. 5, which had a yield point of 47 000 lb. per sq. in., until it had elongated 10% of its original length. This raised its yield point to 71 000 lb. per sq. in., while its ultimate tensile strength was raised only from 83 000 to 88 500 lb. per sq. in. The columns of this steel, No. 6, with the yield point of 71 000 lb. and an ultimate of 88 500 lb. per sq. in., were stronger (in the maximum case more than 10% stronger) than the columns of steel, No. 8, with a yield point of 64 000 lb. and an ultimate strength of 98 000 lb. per sq. in.

Professor Johnson also showed that his parabolic formula agrees well with other tests, notably Tetmajer's.* His (Johnson's) diagrams, comparing his parabolic formula with Tetmajer's tests, were reproduced by Mr. Marston,† with the locus of his (Marston's) formula for eccentrically-loaded columns added, in his discussion of Professor Cain's paper on the "Theory of the Ideal Column." They were again published, as given by Marston, in Professor Cain's discussion‡ of

* The information regarding Professor Johnson's parabolic formula and M. Considère's tests is taken from Professor Johnson's "Materials of Construction," pp. 361–363, and "Modern Framed Structures," pp. 148–152.

† *Transactions*, Am. Soc. C. E., Vol. XXXIX (1898), pp. 108–113.

‡ *Transactions*, Am. Soc. C. E., Vol. LXI (1908), pp. 204–205.

Mr. Prichard.

the paper on "Safe Stresses in Steel Columns," by J. R. Worcester, M. Am. Soc. C. E.

These diagrams are very interesting and instructive. They show a close resemblance between tests and theory when provision is made in theory for the inevitable initial deviation between the axis and the line of thrust; that a parabolic, empirical formula for short columns and those of moderate length is about as good and much simpler than the theoretical one; and that the averages of the tests agree well with the parabolic formula advocated by Professor Johnson; but they also show that a curve which gives more reduction for length accords better with the minimum results.

This was one of a number of facts which influenced the writer, in discussing Mr. Worcester's paper,* to advocate a formula which gives a greater reduction. The formula, the reasons for advocating it, the method of using it in cases of intentional eccentricity, the additional limitations suggested for short columns, and other practical and theoretical considerations were outlined in this discussion and need not be repeated here, but it is best to add a caution with regard to the yield point.

The yield point varies greatly, even in steel from the same melt, as shown in Table 3, and will cause great differences in the strength of columns of ordinary lengths. This is especially important in dealing with columns made of thick plates and large, thick angles, which are likely to have much lower yield points than thinner and smaller sizes of nominally the same grade of steel. It will not do, in such cases, to pin one's faith unreservedly to any formula, no matter how well it is supported by tests of solid bars, $\frac{1}{2}$ in. in diameter, or other small shapes.

Such tests, however, are very useful in determining experimentally the laws of columns by systematically varying one condition while others remain constant. The author's tests are to be commended, in this regard. To plat columns differing greatly in yield point, eccentricity, end restraint, and local stiffness on the same diagram in a kind of hodge-podge, only leads to confusion and tends to discredit the valuable information which has been acquired by analytical and experimental investigation. The value of rational investigation, in which sound theory explains and supplements experiments and experiments put to the test and verify sound theory, should receive general recognition.

An important step in this direction will have been taken when it becomes generally recognized that Rankine's formula is based on a blunder caused by a plausible but fallacious assumption of analogy in deflection of beams and columns, that it has only the semblance of a "theoretic basis," and that it is not even a good empirical formula for wrought iron and steel.

* *Transactions*, Am. Soc. C. E., Vol. LXI (1908), pp. 165-178.

J. S. BRANNE, M. AM. SOC. C. E. (by letter).—The writer has read this paper with much interest, and has been especially attracted by the attempt to find the size of lace-bars to connect the component parts of the columns, in order that said parts may act truly as one piece in the distribution of loading, which produces, generally, both direct compressive stresses and bending stresses, as it may be; and that even wide columns, meaning those of a low ratio of $\frac{l}{r}$ have some side deflection due to compressive loading, speaking now of such deflection in an "ideal column." Mr. Branne.

That there is a natural irregularity and deviation from a straight line, due to lack of uniformity in the component parts, is well known to all engineers, this very thing making the theoretical determination of strength so hard to reconcile with the data as actually revealed by tests.

It seems to the writer that there are two kinds of compression members: the wide ones and the slender ones; the first failing by flow of metal, directly compressed, the second by flow and flexure stresses, which latter are induced by the sidewise deflection of the compression member. By failing is here meant "a perceptible set, showing unfitness for further loading, not actual destruction."

Experiments have shown that wide columns, when properly fabricated, fail when the nominal unit stress approaches or ranges around the elastic limit. The experiments made by Mr. James E. Howard* show test results on wide columns. Just where to draw the line between "wide" and "slender" columns is not so easy. The Progress Report of the Special Committee on Steel Columns and Struts† gives a series of diagrams of tests, which are condensed into one diagram, Fig. 23, showing the results of many tests. The curves are quite flat up to $\frac{l}{r} = 70$, after which they drop more sharply.

The cross-sections of the columns in Mr. Howard's tests were about 90 sq. in., and the ratio, $\frac{l}{r}$, was 26 for Column No. 1, and 47 for Columns Nos. 2 to 5. Columns Nos. 1 and 2 did not fail; but Nos. 3, 4, and 5 did fail, Nos. 3 and 4 in the pin-plates, and No. 5 in the body of the column. The manner of failure in No. 5 was that the webs buckled, resulting in a "sharp bend, or set"; Mr. Howard's Fig. 1, Plate XLII, shows a photograph of Column No. 5 after failure.

In none of these columns was the failure induced to any perceptible degree by flexure stresses, as such combined action of direct compression and bending would have caused failure somewhere below the elastic limit.

* *Transactions*, Am. Soc. C. E., Vol. LXXIII, p. 429.

† *Transactions*, Am. Soc. C. E., Vol. LXVI, p. 401.

Mr. Branne. It must be remembered here that the test pieces of the component parts showed quite a variation in tensile strength, which makes it impossible to refer to a certain magnitude as the elastic limit of the column, as a whole. Columns Nos. 3 and 4 failed at points where the punching of many holes (with much subsequent riveting) induced numerous and complex local stresses; No. 5 failed by the buckling of the web at a point 12 ft. 8 in. from the end, probably due to local weakness, for the test load gave a nominal average unit stress of 30 490 lb. per sq. in. In these columns the lace-bars could not have had much to do before the elastic limit of the column was reached, when locally weak spots put them into activity. It does not seem possible to calculate the lace-bars in wide columns which fail at unit stresses approaching or ranging around the elastic limit. If lacing is used in such columns, it should be quite heavy, in order to take care of local weaknesses; or, still better, lacing should be avoided, and one should use cover-plates or continuous diaphragms, forming part of the section of the column, so as not to waste material.

If heavy lacing is used, the tie-plates at the ends of the columns should be long and thin, rather than short and thick, to steady up the member as much as possible; and where the component parts are fairly wide, say 15 in. or more, diaphragms should be used at the ends, in addition to the plates, thus helping to equalize the stresses carried into the compression member. Similar precautions should be taken at intermediate points where loads are applied.

As to columns of the second kind, namely, the slender ones, all tests show that failure occurs much below the elastic limit, clearly indicating the presence of bending stresses in addition to the direct compressive stress. The bending will induce shear in the columns, producing compressive or tensile stresses in the lace-bars.

If the deflection be assumed to result from a uniform transverse loading, which assumption is not quite correct, it will be evident that the stresses in the lace-bars are greatest at the ends of a free-end column. If the column is not free to move at the ends, but stands between this type and the one with restrained ends, the shear is also greatest at the ends, necessarily, but it is evident that greater care should be taken in this latter type to tie the component parts together more securely at the ends than for free-end columns. If comparison be made, then, with uniformly-loaded beams, it will be noticed that in one with free ends the longitudinal stress caused by the maximum center bending moment, $+\frac{wl}{8}$, has to be transmitted from the compression side to the tension side in a length, $\frac{l}{2}$; in the one with

restrained ends, the same amount has to be taken up (from $+\frac{wl}{24}$ Mr. Branne. at the center to $-\frac{wl}{12}$ at the ends), but this presupposes that the column is held rigidly at the ends, a bending moment, $-\frac{wl}{12}$, is already supposed to exist, and this should be taken care of by extra long tie-plates or diaphragms between the channels.

To proportion the lace-bars under the assumption of uniform transverse loading, a working formula has to be used, for example, American Bridge Company, medium steel, $17\,000 \div \frac{1}{11\,000}\left(\frac{l}{r}\right)^2$, or American Railway Maintenance of Way Association, $16\,000 = 70\,\frac{l}{r}$, and the reduction of fiber stress is assumed as the unit extreme fiber stress due to bending, whence the uniform load can be found.

On this basis of approximating the real conditions, which certainly are unknown, it will be found:

(1) When the column is slender, yet the section not great, the average specification covers the size of lattice-bars very well.

(2) When the column section becomes quite heavy, and, at the same time, due to space requirement of some kind, the component parts, generally channel-shaped, are placed closely together, making failure evident in the lace-bars, it will be found that the generally used lace-bars, as given in the specifications, are not strong enough, but must be increased, finally resulting, economically, in doing away with lacing and substituting cover-plates, or a continuous diaphragm, as noted before.

Finally, as regards the wide column, while the lace-bars can have nothing to do from a theoretical sidewise deflection, at the same time, one component part may be stressed more than the other, due to a faulty foundation or a small error in fabrication, causing an uneven unit stress, which produces stresses in the lacing. As long as the component parts are small, such unevenness can be cared for by the lacing taking up the longitudinal shear; but, when the component parts are very heavy, the lacing cannot make them act as one, for, being proportionally so much lighter than the component parts, they become overstrained and yield. Cover-plates or continuous diaphragms cannot, of course, overcome the faults of foundation or errors in fabrication, but have the strength to transmit unevenness of stress without becoming overstrained, thus making the column act as one homo-

Mr. Branne.

geneous piece, reducing the unevenness of stress in the component parts.

The tests conducted by Arthur N. Talbot, M. Am. Soc. C. E., at the University of Illinois Engineering Experiment Station,* showed an irregular action of the lace-bars, some of the bars showing more stress near the center than at the ends. Further data bearing on these tests may be found in the many interesting notes in the same bulletin.

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* *Bulletin No. 44*, June 6th, 1910.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

A BRIEF DESCRIPTION OF A MODERN STREET RAILWAY TRACK CONSTRUCTION.

Discussion.*

By William J. Boucher, Assoc. M. Am. Soc. C. E.

William J. Boucher, Assoc. M. Am. Soc. C. E.—In 1907, the franchises of many, practically all, of the street railway lines in Chicago had expired, and, through neglect of maintenance, mismanagement, and difficulties in financial affairs, the tracks and equipment were in a miserable and dilapidated condition. By popular vote, new franchises were granted, and by the ordinances of February 11th, 1907, the city became a partner in the street railway business, receiving 55% of the net receipts, and the Board of Supervising Engineers was created. The duties of this Board, as the name indicates, consisted of complete supervision of all the work necessary (including purchase of materials) to place the roadbed, tracks, cars, car-houses, power-houses, and sub-stations in first-class working condition. Mr. Boucher.

The track work was completely remodeled. The first design, known as Type 1 (Fig. 3) was very similar to that described by Mr. Polk. It consisted of steel beam ties, embedded in concrete, which was of greater thickness beneath the ties and rails than between them, where the original earth was left in mounds, undisturbed. The ties were originally placed 4 ft. from center to center. The concrete embedding the ties was brought up flush with the surface of the top flange, which was wider than the bottom flange. These ties are 4¼ in. high, 6 ft. 3 in. long, and weigh 14½ lb. per ft. The tie is used merely to hold the rails down and to line and gauge; it is not depended on to transmit the car load to the soil, that function being performed by the concrete immediately in contact with the rail. A special form of fastening was designed in order to permit the removal of the rail for renewals without disturbing the tie. After the rail was fastened in place on

* Continued from November, 1912, *Proceedings*.

Mr. Boucher. the tie, mortar filling was placed over its flange and upward on each side of the web, and packed under the head of the rail. Tie-rods, with two nuts at each end, pass through the webs of the rails. These rods are 2 by $\frac{5}{16}$ in. in section, which permits paving, of granite or wood blocks, between the tracks, the rods being placed in the usual spaces between the blocks.

Type 2 was a modification of the preceding, and differed from it mainly in the substitution of 6 by 8-in. oak or yellow pine ties, 7 ft. long. Tie-plates are placed under the rails, and the latter are secured to the ties by screw-spikes.

Type 2-A was designed from Type 2, and in it the ties are of wood. They are 3 ft. from center to center, and the excavation has been changed to a uniform section, replacing the mounds of earth between the ties with concrete and permitting the rolling of the excavated trench before placing the concrete. This type is now standard for all new work in all parts of the city beyond the limits of the "Loop" section.

Type 3 (Fig. 3) is used in the business or "Loop" section, where car, vehicle, and pedestrian traffic is very heavy and congested, and where the necessary time cannot be given for concrete to set properly, and also for the reason that the streets in this section are likely to be torn up frequently for conduit, water pipe, or sewer work or subway construction. In this type the sub-grade is rolled and on it is placed 1½- or 2-in. broken stone. This stone is well tamped, on it are laid wooden ties, 2 ft. from center to center, and the usual track structure is built.

For all types the straight track rail weighs 129 lb. per yd. It is a girder rail 9 in. high, with 6-in. bottom flange and ½-in. web. Guard and curve rails are of similar dimensions, but weigh 145 lb. per yd. The chemical composition is as follows:

For the straight rail:

Carbon	0.50 to 0.60	per cent.
Sulphur, not to exceed...........	0.08	" "
Phosphorus, not to exceed.........	0.10	" "
Silicon, not to exceed...........	0.20	" "
Manganese	0.80 to 1.10	" "

For the guard-rail:

Carbon	0.45 to 0.50	per cent.
Phosphorous	0.10	" "
Sulphur, not to exceed............	0.08	" "
Silicon, not to exceed............	0.20	" "
Manganese	0.70 to 1.00	" "

Wood ties treated with preservatives have been used extensively in the newer types of tracks.

All joints on straight track are electrically welded. This work is done under contract by one of the rail-rolling companies with its own equipment mounted on special cars, which run on the new track, and current is obtained from the trolley wire. The ends of the rails are Mr. Boucher.

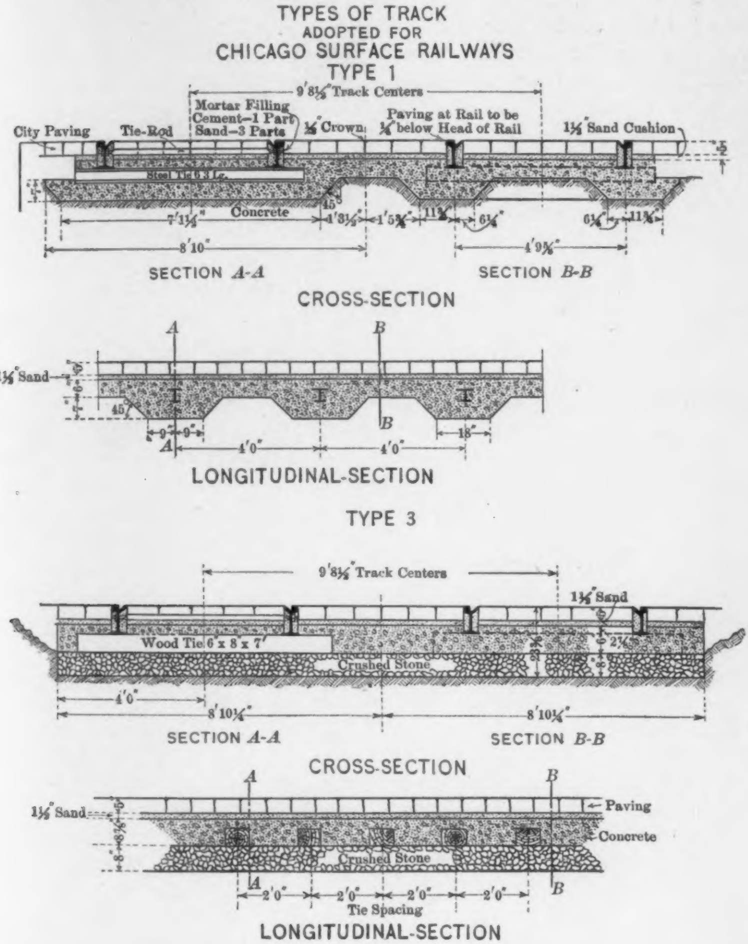

Fig. 3.

butted carefully, but not welded. The weld is accomplished by placing against each side of the webs of the abutting rails, a steel bar, about 1 by 3 by 7 in., on each end of which is a boss, about ⅛ in. high, the object of the latter being to insure good contact against the web. The

Mr. Boucher. bars are held rigidly against the web by heavy jaws operated by hydraulic pressure, and when thus held, a flux is applied and electric current of low voltage and exceedingly high amperage is turned on, the weld being completed in from 3 to 4 min. The heads of abutting rails are afterward smoothed down by a carborundum wheel. The conductivity of the joints must be equal to that of the rails joined.

This construction forms the smoothest running track the speaker has ever ridden over on any street surface railway.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

THE FLOOD OF MARCH 22D, 1912, AT PITTSBURGH, PA.
Discussion.*

By Messrs. Jean de Pulligny, J. Waldo Smith, and Morris Knowles.

Jean de Pulligny, M. Am. Soc. C. E.—The speaker knows very little about the Ohio River, as he only saw it once, but can say a few words about the very severe floods which have to be controlled in Paris, where they caused great damage three years ago. At the present time the speaker understands that the waters are rising again very rapidly, and the people are anxious to know whether they will have to undergo the same severe experience. Mr. de Pulligny.

One of the solutions proposed has been the building of dams up stream. The Seine has several affluents, and the rising of the flood at Paris depends largely on the way in which the various rivers above send down their flow. If the floods arrive at Paris simultaneously, the rise there is very high; if they pass one after the other, the rise is much less, so that keeping one flood waiting a few hours and allowing it to escape afterward might be sufficient to prevent any great rise at Paris.

Besides the construction of reservoirs up stream, other measures have been proposed, and, generally speaking, all those which allow the water to flow freely between the city embankments and below the city may help, to a certain extent, to lower the crest of the flood.

It has been proposed to build, around the city, a deviation canal which would receive the flow of the Marne (an affluent of the Seine) above Paris, and carry it below the city. For helping the flow of waters below Paris, it has been proposed to construct an underground canal from Bougival to Poissy, which would cut straight across the several loops formed by the river below the city. All these works and many others would surely be useful, but, unfortunately, their cost would be very large.

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* Continued from November, 1912, *Proceedings*.

Mr. Smith. J. WALDO SMITH, M. AM. SOC. C. E.—The speaker is not familiar with the Ohio River, or with the circumstances or conditions which control the flow of the Allegheny in the Pittsburgh District, but if these streams are like those with which he is familiar in New York State, New England, and New Jersey, he is convinced that there has been no great flood, but that at some time there will be a greater one, and that there has occurred no great drought, but that at some time there will be a drier one.

To control these floods, and to utilize their waters by storage, is an extremely difficult problem. Floods in the rivers with which the speaker is familiar do not occur by rule; they are likely to occur in any month of the year. He has known of floods in July, in August, in September, in October, in January, March, April, May, and June, and, no doubt, a careful comparison of the records would show that floods had occurred in the other months of the year as well, and severe floods too. In order, then, to utilize the run-off waters for the development of an average power or for augmenting the low-season flow and, at the same time, furnish adequate protection against floods, is a difficult problem.

This discussion is a general one, for, as already stated, the speaker is not familiar with the problems of the Allegheny River, and it may be that floods in that region usually occur in certain seasons only.

Mr. Knowles. MORRIS KNOWLES, M. AM. SOC. C. E.—To the speaker and to others who have studied the conditions in Pittsburgh, this paper is exceedingly interesting. It will be recalled that some criticisms of the Flood Commission's Report have been based on the fact that it was founded on incomplete data regarding rainfall, run-off, and river gaugings; yet this paper is a study, using actual rainfall measurements and stream gaugings on every one of the streams on which the Flood Commission recommended reservoirs. The close correspondence of the results to those calculated on necessarily incomplete data for the Report proves the reasonableness of the Flood Commission's assumptions and justifies its belief that the seventeen recommended projects would prove effective in controlling floods.

Another interesting feature is the light thrown on the one flood which would not have been lower than the damage height, even with the seventeen reservoirs in operation, namely, the flood of 1907. The main causes at the time of that flood were the Kiskiminetas and the Youghiogheny, just as in the flood of 1912, although not exactly to the same degree. This study confirms the view that even these two offending streams could have been controlled (when we consider their time and intensity effect acting with all others) so that conditions would have been materially better.

A word as to the recurrence of floods, and as to the reasonableness of considering any project, not only for this purpose, but for other

Mr. Knowles.

and corelated purposes: During the past 38 years, about 75% of the floods at Pittsburgh have occurred between December 1st and April 1st; the remaining 25% have been scattered through the other months, and, with one or two exceptions, have been small and of short duration. Of this 25% not one would have topped the flood wall which the Flood Commission recommended to supplement the seventeen reservoirs. The reservoirs, therefore, would have been necessary for flood protection only during 4 months of the year, and could have been operated for other purposes during the remaining 8 months. It was this which led the Pennsylvania Water Supply Commission to incorporate such restrictions as the following in two power company charters that have been granted since the Flood Commission made its report:

"That the requirements of the Corps of Engineers, United States Army, in charge of the Allegheny River, as to the minimum stream discharge must be embodied in any plan for using the water stored, as well as the rights of lower riparian owners to have available at all times at least the minimum stream flow, as determined by the Water Supply Commission of Pennsylvania, must be protected.

"That the operation of reservoirs, in so far as the control of floods and the maintenance of low-water flow is concerned, shall be subject to the direction and jurisdiction of the Water Supply Commission of Pennsylvania."

There has been some discussion with regard to the effect on acid conditions. The speaker had hoped that the report of the Flood Commission had made it evident that in this matter, also, there was great benefit to be obtained by this plan. All but four of the proposed reservoirs are located on portions of streams which are not now acid. It is true that some others may become acid, but the location of the reservoirs, near the head-waters, and above most of the mine developments, will restrict this to a minimum. Moreover, the effect at the low-pool dams is no criterion whatever of what will happen in a very large basin with a comparatively high dam. There a sudden rise will not flush out the acid which has accumulated in the bottom, but will dilute it to a greater and greater degree until it overflows the spillway. The same great dilution will take place when a full reservoir is drawn down to prepare for another flood.

The speaker has not the exact figures in mind, but the increased stage at low water on the Allegheny River and the consequent dilution, made possible by the proper operation of the seventeen reservoirs, would result in a reduction of both the hardness and the acidity of that river of something like one-half or one-third at extreme stages, with a marked reduction at other times.

The idea of the beneficial use of such reservoirs for many purposes other than the prevention of floods has resulted, as Mr. Grant has remarked, in the organization of a group of interested people in

Mr. Knowles.

Pennsylvania; some of these are capitalists seeking investment and some are public-spirited citizens and representatives of civic bodies, who have come together on the common ground of desiring a sane development of the water resources of the State. They believe that the doing of one thing does not necessarily antagonize the doing of another, and that many great benefits can be obtained through private development with State regulation and co-operation. They are, therefore, interesting themselves in developing the water laws of the State.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

STATE AND NATIONAL WATER LAWS, WITH DETAILED STATEMENT OF THE OREGON SYSTEM OF WATER TITLES.

Discussion.*

BY KENNETH C. GRANT, ASSOC. M. AM. SOC. C. E.

KENNETH C. GRANT, ASSOC. M. AM. SOC. C. E.—The speaker is particularly interested in this valuable paper because of the direct applicability of the fundamental principles therein stated to the broad policy of water conservation which is now being advocated so strongly in Pennsylvania by the Flood Commission of Pittsburgh and the recently organized Water Utilization Association of Pennsylvania. Mr. Grant.

The Flood Commission was organized by the Chamber of Commerce of Pittsburgh, in 1908, to investigate the causes of, and damage by, the frequent disastrous floods at Pittsburgh, and to study and decide on methods of relief. After four years of exhaustive investigation, for which about $125 000 (obtained from county and city appropriations and by private subscriptions) was expended, the report of the Commission, published in April, 1912, recommended the construction of storage reservoirs on certain tributaries of the Allegheny and Monongahela Rivers. The flood relief which would be obtained by these reservoirs, naturally, would not be confined to Pittsburgh, but would extend to many other communities on the rivers above and below that city. Moreover, the proposed reservoirs would not only provide this flood relief, but would cause an increase in low-water flow which would add greatly to the usefulness of the streams for navigation, water supply, and power purposes.

The Commission pointed out, therefore, that the solution of the problem was so broad in its scope, and so far-reaching in its benefits, that it did not concern the City of Pittsburgh alone, but demanded

* Continued from November, 1912, *Proceedings*.

Mr. Grant. State and National consideration and co-operation. The decisions to which these investigations have led have been aptly stated by the author when he says:

"Some interstate legislation is absolutely essential if the highest development of our streams is to be accomplished. Each river, from its head-waters to its mouth, should be treated as a unit, regardless of State lines."

The Water Utilization Association of Pennsylvania was formed for the purpose of framing and obtaining legislation which should bring about the fullest development of the water resources of the State, and at the same time preserve and dedicate the benefits of such development to all the citizens of the Commonwealth.

This Association and the Flood Commission have given careful consideration to the same thought as that embodied in the author's statement:

"Soon the State and Nation must join in the storage of water for the control of floods and to aid navigation. This water, in passing down the stream, will benefit many private power projects, and these should be compelled to contribute to the cost in proportion to the benefits received."

These bodies, in their work, have also in mind the natural converse of this statement, that private water-power companies, proposing to construct large storage works which will assist in providing flood relief and will improve the rivers for navigation, water supply, and water-power purposes, should receive corresponding co-operation and assistance from the State and National Governments. There are instances in Europe where private water-power companies have been assisted in the construction of their storage reservoirs by the Government in return for a certain amount of additional storage capacity to be constructed and kept empty for flood control. Private users of water for domestic or industrial supply, or for power, located below such large storage works, should also contribute to the cost of their construction in proportion to the benefits received. If effective legislation providing for such co-operation can be framed and enacted, it will ensure the fullest development of the water resources of the State.

Legislation tending toward this end has in fact already been passed in Pennsylvania. At the last session of the Legislature, a bill, drafted by the Flood Commission, was introduced and passed, enabling counties to borrow and expend moneys for the construction of works for flood relief, and also to enter into contracts with each other, or with the State, with other States, or with the United States, for the purpose of carrying out the necessary works.

It is hoped that legislation broadening the powers of the Water Supply Commission of Pennsylvania may be obtained at the coming

Mr. Grant.

session of the Legislature. This Commission, in operation since 1905, is charged with obtaining such complete knowledge of the water resources of the State as shall enable it to provide for their most equitable distribution. It has power over the granting of charters for water-supply and water-power companies, but has no authority where the water is taken by corporations for their own use, by private individuals, or by municipalities. Its effectiveness, in carrying out the purpose named in the act creating it, therefore, is considerably restricted. Its jurisdiction should be widened to cover all users of water.

Within the limits of its powers, the Water Supply Commission of Pennsylvania has done most admirable work, some of it along lines similar to that of the Board of Control described by the author; for example, the work of the Oregon Board for the protection of the public interest, of which the author says: "Public interest demands that water be put to the highest use. * * * an application for either irrigation or power can be denied if it is in conflict with the higher use for domestic supplies," is identical in principle with the attitude of the Pennsylvania Commission in considering applications for charters. There are instances in Pennsylvania where prior applications for charters for water-power companies have been refused on the ground that the stream involved was needed for domestic supply. There are also cases where applications for charters for water companies for domestic supply have been refused, because they interfered, perhaps purposely, and certainly needlessly, with proposed water-power projects.

Charters for water-power companies have also been approved, with certain conditions. For instance, the charters for a large water-power project in Western Pennsylvania were approved recently, on certain conditions, notable among which were the protection of the interests of navigation and of riparian owners below the dam by the maintenance of a suitable minimum stream flow, the presentation, within 12 months, of data showing the extent to which the proposed reservoirs can be used to ameliorate floods, and the placing of the operation of the reservoirs, in so far as the control of floods and the maintenance of low-water flow is concerned, under the direction and jurisdiction of the Water Supply Commission. Had a similar commission been in existence in West Virginia, the plans of a large water-power project, recently chartered and about to begin actual construction, might readily have been enlarged and modified, to the great advantage of both the water-power company and the general public.

A thorough knowledge of stream flow throughout the State is also being acquired by the Water Supply Commission, through the operation of a large number of gauging stations. A complete collection of all existing stream-flow data, some of the records extending back for many years, is now being compiled, and will shortly be published.

Mr. Grant. The Commission has also collected and filed complete statistics of all water and water-power companies in the State, including municipal water supplies. These data are also briefly shown on large-scale county maps of the State, which are convenient for reference in studying the relation of proposed to existing uses of water in a given region. It is evident, therefore, that in Pennsylvania, there is already in existence a body which, if given larger powers, can effectively protect the public interest and assure the most equitable distribution of the waters of the State.

Referring again to the treatment of a stream as a unit from its source to its mouth, there are several interesting and instructive examples of this broad policy in Europe, which the speaker has had the opportunity to examine on two occasions during the last few years.

The largest stream thus treated is the Ruhr River, which empties into the Rhine on the right bank at Ruhrort, in Western Germany. This stream has a length of 143 miles from source to mouth, and drains an area of 2 041 sq. miles. It flows through the great industrial region of Germany, around Essen and Mülheim, and is extensively used as a source of domestic and industrial supply and for power. The demands on the stream became so great that during low water the supply threatened to be inadequate. In 1897, after long deliberation and much difficulty, a voluntary association was formed by the users of the Ruhr water for the purpose of improving the flow of the river. The membership of this association includes cities, factories, water-power companies, etc. There are eleven dams on the tributaries of the Ruhr, ten of which were built by smaller associations of the water users on the respective tributaries, and one, now nearing completion, by the Ruhr association. These smaller associations were formed under a Prussian law. After two-thirds of the water users below the proposed dam have agreed to form such an association, the other third is obliged to join with them. The main association of the Ruhr, called the *Ruhrtalsperrenverein,* was formed voluntarily, without such a law; but it can compel users of water from the Ruhr to pay into the association, in the case of water-power plants, according to the head and the quantity of water used, and, where water is taken for domestic or industrial supply, according to the quantity used. The assessments on the members of the smaller associations on the tributaries are made in a similar manner. The main association of the Ruhr pays about \$70 per annum per 1 000 000 cu. ft. of storage capacity of the reservoirs to each of the ten associations on the tributaries.

Another example of similar co-operation in river regulation is to be found on the Wupper River, the next tributary of any size emptying into the Rhine on the right bank above the Ruhr. It has a drainage area of about 240 sq. miles, and flows through the thickly

populated manufacturing region around Barmen and Elberfeld. Before the construction of reservoirs on its tributaries, the stream had a very irregular discharge, varying between 0.05 and 90 sec-ft. per sq. mile of drainage area. Flood damage was frequent and considerable, and the numerous water-power plants suffered greatly from low water. Mr. Grant.

After long deliberation, an association for the construction of reservoirs in the drainage area of the Wupper was formed under a special Prussian act. This association has built three reservoirs on tributaries of the Wupper, for the control of floods and the increase of low-water flow. All water users below these dams must pay an annual assessment to the association.

A third association, of similar character, has constructed six reservoirs in the Görlitzer Neisse, near Reichenberg, in Bohemia. This stream rises in Northern Bohemia and flows northward for 124 miles, emptying into the Oder River about 15 miles below Crossen, in Prussia. Its valley receives a heavy precipitation in its upper portion (13.5 in. in 24 hours having been recorded at some points) and has been repeatedly devastated by floods; therefore, after the great flood of 1888, an association was formed to plan and carry out the construction of protection and regulation works, consisting of widening and straightening the channel and raising and protecting the banks. The estimated cost of these improvements was so heavy, and their probable effectiveness in a great flood so doubtful, however, that practically nothing was done by the association, the actual work confining itself to repairing damages and building the bank protections most urgently needed by the individual property owners.

In July, 1897, this part of Europe was visited again by devastating floods, which revived public interest in flood relief to such an extent that a convention, in which all the neighboring cities and towns were represented, was held in Reichenberg in the fall of that year. At this meeting it was decided to investigate the feasibility of constructing reservoirs for flood control. In January, 1901, the preliminary studies were sufficiently complete to establish the general plans, which contemplated the construction of six reservoirs in the neighborhood of Reichenberg, controlling the run-off from about 29 sq. miles, and in critical flood time holding back about 3 530 sec-ft. of damaging flood discharge.

The result of the investigations gained the association many new supporters, and assured the sympathy of the population of the entire valley of the Neisse with the project. In fact, one of the most noteworthy features of this undertaking is the widespread interest it aroused in the surrounding country and the universal financial support it received. Although all the reservoirs are located in Bohemia, the benefits, both in flood control and increase of low-water flow, are

Mr. Grant. felt by the Saxon and Prussian interests along its lower course, and these two countries, together with various cities, communities, and private interests, co-operated with Bohemia in their construction. The total cost of the work was $1 320 000, and the following contributions show the extent of the co-operation:

Bohemian Government	$660 000
Prussian Government	38 400
Prussian Province of Silesia	9 600
Prussian County of Ober-Lausitz	14 400
City of Görlitz (Prussia)	14 400
Saxon Government	24 000
Combination of Saxon and Prussian water interests	72 000

It is also of special interest that the users of water for power development from three of the reservoirs pay $12 per h.p., and from the other three reservoirs, $28 per h.p. per year. Three of the dams for these German reservoirs are shown on Plate CXXXIX.

As an instance of what such co-operation might accomplish in the United States, the conditions in the Beaver Valley in Western Pennsylvania may be mentioned. This stream drains about 3 040 sq. miles, and empties into the Ohio from the north about 25 miles below Pittsburgh. It is extensively used as a source of domestic and industrial supply, as it flows through a thickly-populated manufacturing district. It is also used considerably for power. On its upper waters there is an ideal reservoir site where a low earthen dam would create a reservoir of large capacity, overflowing a large tract of useless swamp land. This site has already been studied as the source of feed-water for the proposed Lake Erie and Ohio River Canal. It has also just been carefully surveyed and mapped by the State Water Supply Commission, under a special appropriation by the Legislature, for the purpose of determining the possibilities and best use of the site. Here is an ideal condition for a *Beavertalsperrenverein,* which could be made up of the water users in the Beaver Valley and the Canal Company, with the possible addition of the communities along French Creek, a large northern tributary of the Allegheny, because of the flood control that would be afforded by a second reservoir proposed by the Canal Company on the north branch of this creek, which would impound the flood-waters of that stream and deliver them through a canal into the reservoir on the Beaver River head-waters.

Another example of what might be done by such co-ordination of interests is afforded by the Youghiogheny River, a stream rising in Northern West Virginia, and entering the Monongahela 15 miles above Pittsburgh. This stream is used extensively as a source of industrial and domestic supply, and has several favorable sites for water-power

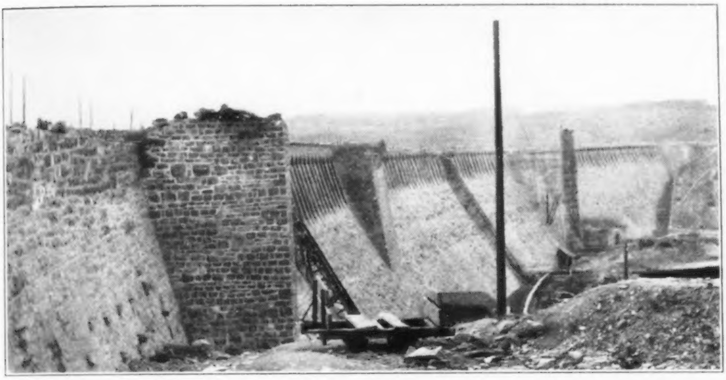

Fig. 1.—Dam Built by Association for Improving Low-Water Flow of Ruhr River, Germany.

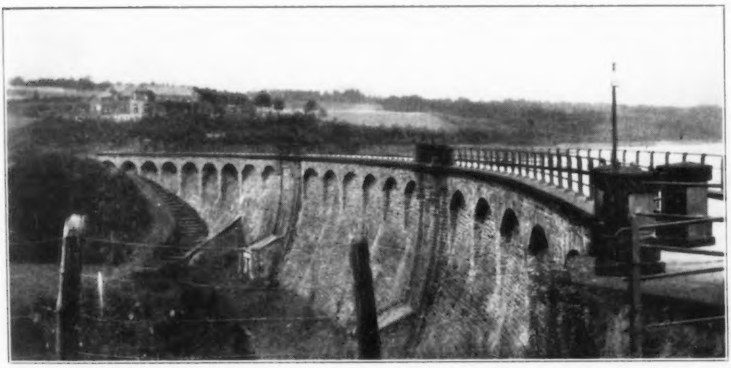

Fig. 2.—Dam Built by Association for Controlling Floods and Increasing Low-Water Flow of Wupper River, Germany.

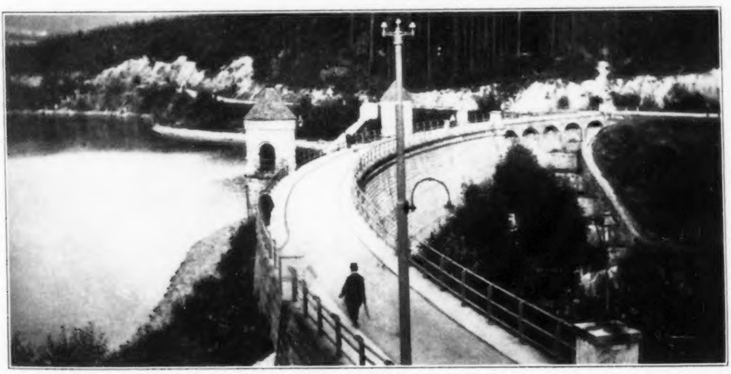

Fig. 3.—Dam Built by Association for Controlling Floods and Increasing Low-Water Flow at Reichenberg, Lohemia.

Mr. Grant.

development. The Flood Commission of Pittsburgh proposes to store part of its flood run-off for the control of floods and for the improvement of the low-water flow, this stream being one of the chief offenders in Pittsburgh floods. The United States Government proposes to slack-water the lower 19 miles, to provide navigation up to West Newton, although a study of the low-water flow indicates that a pool-full stage could not be maintained without the assistance of additional water from storage reservoirs during dry weather. The co-operation of all these interests in the construction of storage reservoirs for the regulation of the flow of this river would unquestionably be of great benefit to all concerned. It would make feasible the development of a large quantity of water power which cannot be developed economically if the entire cost of the storage works must be borne by the water-power interests.

A third example may be found in Eastern Pennsylvania, where a large water-power project in operation on the lower Susquehanna River could greatly increase its capacity if the low-water flow were increased by storage reservoirs on the upper waters. The value of such increased flow to this one power plant would not pay for the cost of constructing such reservoirs; but if other plants, made feasible by the increased low-water flow, were constructed, and contributed to the cost of the storage works, and if co-operation were also obtained from the communities, counties, railroads, and other interests damaged by floods, such a broad treatment of the river would undoubtedly become feasible.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

THE SEWICKLEY CANTILEVER BRIDGE OVER THE OHIO RIVER.

Discussion.*

By Messrs. L. J. Le Conte, Charles Worthington, Theodore A. Straub, and C. W. Hudson.

L. J. Le Conte, M. Am. Soc. C. E. (by letter).—There is apparently nothing whatever to show why the cantilever type was selected for this simple bridge site. As far as one can see by the plans, there are no local physical conditions which call for it. It seems to the writer that three simple spans, approximating 450 ft. each, would cover the ground and furnish a stiffer, far better, and cheaper structure in every respect. He ventures to say, moreover, that the probable saving in first cost would have been fully 25%, if not more; consequently, he naturally cannot see the propriety of building such a structure at such a site. Mr. Le Conte.

It is natural to presume, therefore, that the taxpayers of Sewickley Borough and Moon Township, who paid for it, have substantial grounds for complaint.

Charles Worthington, M. Am. Soc. C. E. (by letter).—The writer would like to inquire what provision was made in this bridge to take care of the secondary stresses which develop in a cantilever truss of this type? Mr. Worthington.

In the old Quebec Bridge, these secondary stresses were of such magnitude as to destroy the structure when the direct or axial stresses were about one-half the elastic limit of the material.

In the Beaver Bridge,† a very expensive roller bearing was provided under the main pier supports to reduce the secondary stresses.

* This discussion (of the paper of A. W. Buel, M. Am. Soc. C. E., published in September, 1912, *Proceedings*, and presented at the meeting of November 20th, 1912), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† "The Pittsburg and Lake Erie Cantilever Bridge over the Ohio River at Beaver, Pa.," by Albert R. Rayner, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXIII, p. 136.

Mr. Worthington. In the Sewickley Bridge the members seem to have been proportioned for the axial stresses only. The writer has investigated a few of the members shown on Plate LXXXII, and finds that the sections agree pretty closely with those required by the stresses given on that plate and the unit stresses given in the body of the paper.

Consider, for example, the bottom chord member, L_{10}-L_{11}. Its length, l, is 900 in., and its radius of gyration, r, is 9.22 in. The unit stresses, as determined by the author's text, are then as follows:

For live load, $12\,000 - 40 \times \frac{900}{9.22} = 10\,040$ lb.

For dead load, double this, or 20 080 lb.

For live load, dead load, and wind load combined,

$10\,040 \times \frac{20}{12} = 16\,750$ lb.

So that the required sectional area of the bottom chord member, L_{10}-L_{11}, is as follows:

Dead load.........	1 467 400	at 20 080 =	73.0	sq. in.
Live "	623 600	" 10 040 =	62.0	" "
			135.0	" "
Wind "	966 100			
Total	3 057 100	at 16 750 =	183.0	sq. in.

Taking the greater of these required areas, 183.0 sq. in., it will be seen that the actual area of 190.2 sq. in., is only 4% greater than that required, so that practically no excess of material has been added to take care of the secondary stress in this member, and the only provision for it lies in the unit stresses themselves. The writer does not think that 24 000 for dead load and 12 000 for live load, properly reduced for columns, are low figures for direct stress in steel of the character probably used in this structure.

The eccentric used at the L_0 point is in itself a source of secondary stress. If this eccentric were to be fixed in position so that the offset of centers of ½ in. lies in a horizontal direction—a very possible condition—there would be developed at this point a bending moment of $240\,000 \times \frac{1}{2} = 120\,000$ in-lb., which, measured in terms of the area of the two 10 by 1½-in. bars multiplied by the stress in the extreme fiber of bar, is $120\,000 \times \frac{6}{10} = 72\,000$ lb.

The axial stress in these two bars is constant at 384 000 lb., so that this secondary stress of 72 000 lb. amounts to some 18.7% of the axial stress. This secondary stress may not develop while the bridge is new, on account of the lubrication applied to the eccentric, but

it will certainly develop later. Taking the coefficient of friction between the pin and the bar at 0.4, the resisting moment to be overcome before the bar would rotate on the 9¾-in. pin is, $384\,000 \times 0.4 \times 4.87 = 750\,000$ in-lb., or about six times the amount necessary to develop the moment of 120 000 in-lb., due to eccentricity in application of the load to these bars. Mr. Worthington.

Theodore A. Straub, M. Am. Soc. C. E. (by letter).—The results obtained from the methods used in the construction of the Sewickley Bridge were so satisfactory that a few words from the writer may be of interest. In all the departments of the Fort Pitt Bridge Company—the drafting-room, shop, and erection—the results were economical in the broadest sense of the word. These methods also aided materially in establishing that complete confidence and hearty co-operation which is so desirable and effective in the execution of work of this kind among all persons connected with it. Mr. Straub.

Duplication of structural steel members cannot often be controlled, but it is frequently possible to duplicate the parts which compose such members, even if there is a marked difference in their final make-up. This may or may not be the duty of the purchaser's engineer, but, if it is, the latter, by the stand he takes in settling questions of detailed shop drawings, can often make a seemingly inexpensive piece of work very costly to the fabricator, or *vice versa*. It will be noted that a special effort was made to secure such duplications, and the consequent economical results were due to the latitude allowed to the Bridge Company by the County Engineer.

Where possible, all field connections were reamed in their final relative positions in the shop. This resulted in securing good fits and finish, and obviated the necessity of the correction of mistakes in the field. The writer might say, in passing, that this is the general practice of the Fort Pitt Bridge Works on all work of any magnitude, and it has always been felt that it is an economical method of procedure. All field connections were made in the manner planned, without trouble and interferences, the final connections and adjustments being especially satisfactory.

The writer believes that the special angle lacing bar, as described by Mr. Buel, was used for the first time, as such, in an important bridge member. In sufficient quantities its fabricating costs are not excessive, and the bars can be made in any blacksmith's shop which has a power hammer or press. This lattice bar is compact, efficient, and neat in appearance; it also has the additional advantage of readily shedding water.

Referring to Plate LXXXVII, entitled "Profiles for Erection": These data were used freely, and were found to be especially helpful throughout the erection of the structure. They proved a ready check

Mr. Straub. at all stages, and satisfied the engineers and erectors at all times that the work was progressing properly and safely.

The simple and inexpensive anchorage arrangement required the efforts of one man only in making the adjustment, thus giving extremely satisfactory results.

Both the wing and creeper travelers responded at all times to the duties planned and imposed on them in a very economical and efficient manner. It was also found that the adjustable features of the wing traveler, as well as the extra weight, did not affect the cost of its operation materially. The Bridge Company anticipates its use for future erection, and considers it a very good and efficient tool.

Complete harmony existed at all times among the engineers of the County and of the Bridge Company, and this was not the least of the pleasing results obtained by the methods used. All points in question were discussed freely and openly, and were settled promptly. Much credit is due to all in this regard and they deserve praise.

Mr. Hudson. C. W. Hudson, M. Am. Soc. C. E.—It would be of interest to structural engineers to know how much it cost to make the anchor arms of this bridge self-supporting in case of a wash-out of the falsework. This expense, whatever it amounted to as a percentage of the whole cost, was in the nature of insurance against the loss of these arms, and against the loss due to the consequent delay in completing the structure in case of such wash-out.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

PORTS OF THE PACIFIC.

Discussion.*

By Messrs. L. J. Le Conte, E. P. Goodrich, and Lewis M. Haupt.

L. J. Le Conte, M. Am. Soc. C. E. (by letter).—On page 1118† the author speaks in glowing terms of the future possibilities of the Lake Washington Ship Canal scheme, now well under way. As a matter of historical interest, it may be stated that, in October, 1871, the late Gen. B. S. Alexander, accompanied by Lieut. (now Col.) Thomas H. Handbury, M. Am. Soc. C. E. (both of the Corps of Engineers, U. S. A.), and the writer, as Assistant Engineer, went to Puget Sound with the view of making extensive "current observations" for the purpose of determining the practicability of harbor defense by torpedoes. While on this mission, Mr. Briar Brown and Dr. Whitworth, two estimable citizens of Seattle, took the party to the proposed ship canal site, and explained the scheme. Orders were received to make the survey, and called for plans, estimates of quantities, and probable cost of making a ship canal, Mr. Le Conte.

"with the view of ascertaining the adaptability of this lake for a naval depot, and the proper route for a ship canal to connect it with Admiralty Inlet, and the cost of such canal.‡"

The field work was done by Lieut. Handbury and the writer. The report of the results of this survey will not be found in the River and Harbor Index, because the expenses came out of "Surveys for Military Defense"; consequently, it will appear only among the Senate Executive Documents for the fiscal year ending June 30th, 1872.

* This discussion (of the paper by H. M. Chittenden, M. Am. Soc. C. E., assisted by A. O. Powell, M. Am. Soc. C. E., published in September, 1912, *Proceedings*, and presented at the meeting of November 20th, 1912), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† *Proceedings*, Am. Soc. C. E., for September, 1912.

‡ Report, Chief of Engineers, U. S. A., 1872, p. 26.

Mr. Le Conte.

At the time this survey was made, it was claimed that the main object of the fresh-water depot was to enable the naval fleet to go up into Lake Washington, and, while taking on stores and supplies, all barnacles, sea-grasses, etc., befouling their bottoms would naturally drop off, leaving them clean when they proceeded to sea. In ordinary cases, of course, ships would have to go into dry dock to have their bottoms cleaned. For large vessels, this would be a matter of $2 000 per day in dock expenses, hence the great saving to be naturally expected. The writer has no doubt whatever of the great future possibilities of the Lake Washington Ship Canal, and will watch its growth in usefulness with interest.

Speaking of the Sound waters in general, the writer was greatly impressed by the bold shores everywhere. In fact, a 3 000-ton ship can go up to the shore, make fast to the trees, and lay there as long as necessary. This same feature was also noted in Lake Washington, and the writer is firmly of the opinion that it is due to vertical stratification; thus, whenever one stratum drops off, it always leaves a new vertical face standing. This natural feature makes trouble for the commercial wharves along the shore, but, nevertheless, it has its advantages in some cases.

The removal of Blossom Rock, in San Francisco Harbor, was a most interesting piece of work, and the fact that it ended disastrously was due, as usual, to unexpected contingencies. The writer had the pleasure of visiting the work several times during its progress, and just after the blast, in April, 1870, was told by the late Col. A. W. von Schmidt, the contractor, that he was compelled to set it off some time before he really intended, because of the exceedingly shaky condition of the rocky shell overhead, which had to be propped up with timbers. The tremor due to the working of the hoisting engine at the main shaft would start up leaks everywhere, and he was dreadfully afraid of a general collapse; hence the blast was premature. The subsequent quantity of dredging and scraping necessary to get the required depth of 24 ft. at low water ate up all the profits on the contract.

The draft of vessels entering the harbor gradually increased to 28 ft., and in November, 1902, a second contract was let for increasing the depth over the rock to 30 ft. at low water. The extra 6 ft. to be removed being largely loose broken stone, the contractor resorted to surface blasting, and excavating with a clam-shell dredge and a 10-ton bucket of the grapple type with long lever arms. The results were highly satisfactory.

The same system and dredge were also used in the removal of Shag Rock No. 2, and the results were equally satisfactory. The whole secret of success in surface blasts lies in the use of small charges, exclusively. The writer had local charge of the removal of Rincon Rock, in San Francisco Harbor, and the contractor engineer,

Mr. Albert Boschke, made the great mistake of using 40-lb. charges for surface blasting before dredging the broken rock. Such large blasts simply expend their force in making a fine fountain, and the mechanical effect on the bottom, where it is needed, is practically nothing. Mr. Le Conte.

The quantity of powder required for each blast is that which will just begin to make a fountain. In most cases this is about 6 lb. of, say, 90% nitro-gelatine. These small blasts, judiciously distributed and followed immediately by clam-shell dredging, are by far the cheapest and most effective, where applicable.

A few remarks about the great jetty at the mouth of the Columbia River may not be out of place. The prodigious movement of sand mentioned in the early reports on this work proved to be a myth, and later experience showed that the sand simply traveled in an orbit of limited diameters. When the Point Adams Jetty was well extended, the sand movement practically disappeared, because it was "bottled-up." The depth of 31 ft. was obtained over the bar, and everybody rejoiced. Peacock Spit, on the north side, next to Cape Disappointment, was practically the natural north jetty. Unfortunately, Peacock Spit was cut away by subsequent heavy storms and strong currents, and as a result the bar shoaled up to its original depth of 21 ft. at low water. It is very evident that the only thing to do is to build another jetty from Cape Disappointment out along the crest of Peacock Spit, and thus maintain the spit and hold the tidal currents up to their work along and against the south jetty. The writer is of the opinion that a 31-ft. channel cannot be maintained unless Peacock Spit is fixed on the north side.

Dredging has grown to be a most important adjunct to every type of harbor work. The great advances made in machinery have cheapened the cost of dredging to such an extent that all schemes for proposed harbor work are now materially affected thereby. It may be stated that the day for extending jetties out into deep water in order to maintain the depth on the bar is gone forever The annual interest on the cost of deep-water extensions will pay for the necessary dredging five times over.

The improvement of tidal flats for commercial purposes is looming up everywhere. When the fact is considered that the new property, after full improvement, can be readily sold for $30 000 per acre, one can see how easy it is to pay for proposed schemes.

Everywhere in Europe it is customary for the government to condemn and purchase all property contiguous to a proposed harbor work, and, after improvements are completed, sell the property to the highest bidder. As a result, it not only gets all its money back, but in addition a handsome profit. The writer fails to see why the same cannot be done in the United States.

Mr. Le Conte.

The work for bar dredges is growing more important every day. Where the entrance is largely under good shelter, the work can be done for 4 or 5 cents per cu. yd.; but where the bar is exposed to heavy deep-sea swells a large part of the time, the working period is cut down fully one-half, and the cost naturally goes up to 8 and 10 cents per cu. yd. These results are so flattering that harbors of any importance cannot afford to get along without a good bar dredge for general use, as it will pay back its first costs in one year's operation.

Mr. Goodrich.

E. P. Goodrich, M. Am. Soc. C. E.—Having been a student of port problems for some years, and being intimately connected with some of the work described in this paper, the speaker has studied it with considerable attention. Concerning several points, his interest has been specially aroused. The general problem of the Panama Canal and its effect on the commerce of the world is of great interest, and, together with the various physical problems discussed by the authors, has been of professional interest to the speaker. On these matters, however, he does not care to make any comments.

Concerning several non-professional points, the speaker's interest has been similarly aroused. Among these is the fact that, in a professional paper before the American Society of Civil Engineers, nothing but an obviously exaggerated newspaper report has been used in describing the port conditions at Los Angeles, Cal. The speaker may lay himself open to criticism for possessing unprofessional curiosity, but he must confess that had it been possible, he would have liked to await the authors' final discussion in order to ascertain whether or not they would correct the descriptions of the projected Los Angeles work, by making use of modified but obviously more accurate subsequent newspaper articles, to the files of which they evidently have access. The speaker is in entire accord with the opinion expressed in the paper, that it is possible that the popular enthusiasm which is now working such wonders on the Pacific slope may go too far, and that the pendulum may eventually start on a reverse swing. Thus, both in Portland, Ore., and in Los Angeles, Cal., he has laid himself open to the possibility of becoming unpopular by sounding a strong note of conservatism. It may not be out of place to state that the newspapers in both cities reported such fact, which might have been discovered by the authors had they read the local papers carefully. Because of the juxtaposition of the speaker's name, coupled with certain comments about the Los Angeles work, with the authors' reference to "shrewd promoters of port development and no less shrewd port engineering experts," who "will doubtless make the most of the present opportunity and ride on the crest of the flood wave to a point which the normal depth of water would not permit them to reach," he is afraid that casual readers will obtain an erroneous conception of the Los Angeles work and of his connection with it. Even the qualifications

inserted by the authors are considered inadequate to meet the exactions of truth, and, therefore, the following facts are presented to the Society in an effort to secure justice for a city which the authors themselves have characterized as "incontestably the center of activity of Southern California" and destined to "become a great port, not because Nature made it so, but because her own virile people have said so." Mr. Goodrich.

The facts (which might have been ascertained by correspondence or by perusal of public prints) are as follows: The City of Los Angeles owns about 200 acres in the outer harbor, comprised in two tracts of about 150 and 50 acres, respectively. The outer harbor works now contemplate the immediate improvement of only one of these tracts, work on the 50-acre tract being practically under contract to the extent of bulkheading and filling, while plans for the 150-acre tract are in process of evolution, the detailed design being made subject to the determination of certain physical conditions with regard to currents, the occurrence of rock in the foundations, and the securing of certain real estate to provide rights of way for means of access.

In a professional paper* published nearly a year ago by Amos A. Fries, M. Am. Soc. C. E., Captain, Corps of Engineers, U. S. A., a suggestion was made with regard to the possibility of further improvement of the outer harbor, proposing a tremendous breakwater and certain long piers similar to those described by the authors and ascribed to the speaker:

"* * * due to the fact that the ocean bottom becomes flatter as you go east from Point Fermin, the breakwater can be extended to inclose any amount of anchorage that may be desired. Indeed, a 25 000-foot extension, should that much ever be required, could be made on almost the same line as the outer arm of the present breakwater, and while keeping in depths averaging barely 48 feet would inclose 10 square miles of water, half of which would average more than 36 feet.

* * * * * *

"Not only can the breakwater be greatly extended, but if fifty or one hundred years hence a long extension becomes necessary, the harbor frontage itself can be increased at least 17 miles by the construction of nine slips between Deadmans Island and the entrance to the Long Beach Harbor. The slips beginning at the present 18-foot curve could be made in lengths varying from 6 500 to 2 100 feet, with a tongue of land 1 000 feet in width between each two slips.

"It would seem advisable, whenever any considerable extension of a breakwater is made, to leave a gap 2 000 feet in width between the present breakwater and the beginning of the extension."

* "Los Angeles Harbor," *Professional Memoirs*, Corps of Engineers, U. S. A., and Engineer Department at Large, Vol. IV, No. 13 (Jan.-Feb., 1912), p. 1.

Mr. Goodrich.

While the authors state that great reliance is being placed on the inner harbor in the development of port facilities, they also say that:

"A great saving fact in the enormous labor of building the Harbor of Los Angeles is its close relation to the industrial development of the city. The lands into which the waterways are being dredged are admirably adapted for factory locations, and the material of excavation is being used in making the necessary fills. The whole development goes hand in hand in a way to produce the best results."

It is the speaker's firm opinion that the inner harbor will be developed almost exclusively for industrial plants and the outer harbor for deep-sea tonnage. At the wharves of the inner harbor, vessels with full cargoes for special manufacturing plants will berth. In order to make the best use of certain dredging which has already been completed, the speaker recommended, and it is understood that the contracts have now been let for, certain temporary wharves located on one of the arms of the inner harbor as now laid out. Only a portion of this wharf will be shedded, and this whole improvement will be carefully studied in an effort to determine what is likely to be the future of that portion of the port. It is further believed that the authors should have put their qualification with regard to the profitableness of reclaiming tide flats, noted under the head of "Dredging," in closer proximity to the point they endeavor to make of the great cost of doing the harbor work in Los Angeles. They say:

"Now that the reclamation of tide flats is becoming so profitable an enterprise, dredging will be resorted to more than ever, the operation serving the double purpose of excavating slips and channels and filling the abutting lands."

Increased real estate values will be much greater in Los Angeles in comparison than in many other locations, so that moneys expended will produce larger returns for self-supporting enterprises, such as harbor propositions have generally shown themselves to be, and the criticism of the costly nature of the work at Los Angeles Harbor submitted by the authors becomes largely nullified. In reference to this, it may be well to call further attention to what the authors say with regard to San Francisco Bay:

"The shoaling of San Francisco Bay is one of those great natural blessings which the unthinking are so accustomed to look on as a curse. One-tenth of its natural area, with deep connecting channels, would serve every possible need of commerce, while the other nine-tenths would be of immeasurably greater benefit reclaimed and turned to industrial or agricultural use. Every cubic yard of earth washed down from the rugged slopes of the mountains is worth a thousand times more in those low areas, where it is turned to efficient use in the service of Man."

As to the inner harbor at Los Angeles, the speaker has recommended certain alterations in the harbor lines, a portion of which

proposed changes has been presented to the War Department for consideration, and it is hoped will receive its approval. Again, with regard to the inner harbor, such changes in harbor lines are in close accord with the recommendations contained in the paper by Capt. Fries, who writes as follows: Mr. Goodrich.

"Between Deadmans Island and the turning basin in the inner harbor there are 18 000 feet of bulkhead lines and 500 acres of land, most of which is now reclaimed. The frontage there may be increased a few thousand feet by slips. Above the turning basin, in the east and west basins combined, there are 750 acres of reclaimed land and 52 000 feet of frontage exclusive of the Salt Lake Railroad Company's land on the south and east. This frontage may be increased economically by slips to the extent of 13 000 feet.

"The above is along present approved harbor lines, and while the total—including the slips suggested by the writer and the 12 000 feet along the Salt Lake Railroad Company's property—amounts to 132 000 feet, or 25 miles, it does not represent half the frontage that can be developed if the future shall show that more is needed.

"Bounded by the bluffs of San Pedro on the west, the Anaheim Road and the city of Wilmington on the north, the City of Long Beach on the east, and San Pedro Bay on the south, there are some 8 square miles (about 5 000 acres) of swamp, tide, and submerged lands capable of being practically improved as part of the inner harbor. Before this is all developed it is evident that the present anchorage area will be too small.

* * * * * *

"* * * the land owned by the Salt Lake Railroad Company between the Long Beach Harbor and the east basin can probably best be developed by slips opening into the east basin and the Cerritos Channel between it and the Long Beach Harbor, giving a frontage of 12½ miles. On Plate V the channels are shown 500 feet wide on the north side of Cerritos Channel, where they are about 1 mile in length, and 300 feet wide on the south side where the lengths are 2 000 feet."

The authors are entirely wrong with regard to their criticism of the possibilities of the inner harbor of Los Angeles becoming silted by material carried in the floods of the Los Angeles and San Gabriel Rivers. The speaker cannot do better than quote again from the report of Capt. Fries:

"The question of maintenance of depths is always a very important one when considering the future of a harbor. In this matter Los Angeles Harbor is exceedingly fortunate. Indeed, it is hard to conceive of an ocean harbor that will cost less to maintain. The two causes of the deterioration of a harbor are silt carried down by rivers flowing into the harbor and sand piled up at the entrance by cross-currents and wave action.

"Ordinarily, the Los Angeles River is the only one whose waters reach the harbor during the rainy season. During the greater part of the year the river goes entirely dry before reaching the sea, due

Mr. Goodrich. to irrigation and the great quantities of water used in the city of Los Angeles. It is noted, however, that during the winter of 1910-1911 the San Gabriel River, which ordinarily flows into Alamitos Bay, about 10 miles east of Los Angeles Harbor, broke from its regular channel into one known as New River at a point about 20 miles from the harbor and, following the New River, united with the Los Angeles River at a point about 5 miles north of the harbor. This was one of the worst floods known in many years and carried into the Los Angeles Harbor possibly 350 000 cubic yards of material. Efforts are now being made by railroad companies and agricultural interests in the vicinity of the break to make such improvements in the bank of the river as will confine it in the future to its regular channel, emptying into Alamitos Bay.

"The Government has been asked to aid in this as a measure of protection to the harbor, and steps are being taken in that direction. Unquestionably this improvement will be made, but even if the San Gabriel River should regularly flow into the Los Angeles Harbor, the cost for dredging would still be comparatively small, as the records for nearly fifty years show only five serious floods. These occurred in 1867, 1873, 1884, 1891, and 1911. The Los Angeles River itself carries down some material in smaller floods at lesser intervals, but the amount is so small as to be scarcely noticeable, except just where the river first enters the deep water of the harbor."

Another point described by Capt. Fries, and now being actively pushed by the local authorities, is the widening of the channel between the inner and outer harbors to a minimum of 750 ft. A request for this improvement has been formally filed with the War Department, and it is believed that the Washington authorities look favorably on the suggestion.

In discussing the administrative systems of the ports, the authors criticize the conditions in Los Angeles, where there was originally and is now technically a divided responsibility. The trouble described by them, however, has been entirely overcome by the Mayor, who appointed what is called an "Advisory Harbor Commission," consisting of the two boards meeting jointly, with the Mayor as Chairman. During the meetings of this joint board all matters relating to harbor affairs are talked out, and differences of opinion are eliminated. The Mayor's solution of the small difficulty described by the authors has proven highly advantageous to the community.

With reference to the table of comparative costs to ship and to cargo in Pacific Coast ports, the fact should be pointed out that, with the exception of San Francisco, the table shows Los Angeles to have the lowest total cost. It should be noted further that the handling rate of 41.8 cents has been taken from an average which might be vastly different in an actual case. The dropping of the 0.8 cent would reduce the cost by nearly $200.

The speaker is extremely sorry that the authors were not more accurate in their statements with regard to Los Angeles Harbor, and

believes that it throws grave doubt on the accuracy of the paper in other points. Mr. Goodrich.

Lewis M. Haupt, M. Am. Soc. C. E. (by letter).—This is a very comprehensive paper, not only on the engineering problems of Pacific Coast harbors, but also on their commercial relations; and, in view of interesting discussions previously published in the *Transactions* of this Society,* the present contribution is one of great value to the Profession as a guide to and aid in the early solution of these intricate questions. Mr. Haupt.

History and experience are the foundations of theory on which the engineer must rely to shape his course in order that he may best serve his Profession and his country by removing these obstacles to international commerce, hence it is that this paper is peculiarly appropriate on the eve of the completion of the Panama Canal. As it covers all the important ports from San Diego to Vancouver, no detailed discussion is possible, nor are the maps appended sufficient for such purpose; the text, however, supplements them in large part by stating local conditions and results, and several extracts therefrom are submitted as pointing to such modifications of practice and policy as to give promise of much greater certainty of securing early results.

As it is not possible to review all the cases cited, that of the typical and important Columbia Bar is taken as an example, because, as the authors very justly state:

"This work * * * is probably the largest and most difficult of its kind ever attempted * * *. The two great obstacles to be overcome are the storms and the teredo * * *. The roughness of the sea precludes the use of barges for dumping rock, thus necessitating a trestle, and the trestle piling is the particular delight of the teredo which puts it out of commission in from 10 to 20 months. * * * the embankment is progressively shaken down during each winter season."

Specifically, the work is described as follows:

"The jetty was to be built of large size stone on a brush mattress and raised to the level of low tide. Later, the project was changed to raise it to high tide, and four groins were to be built from the north side to arrest scour. Work was begun in 1885 * * *, and was completed ten years later at half the estimate. The depth on the bar had increased, in the meanwhile, from 21 to 31 ft., and the work seemed to have accomplished its purpose perfectly.

* * * * * *

* "On the Straits of Juan de Fuca, Puget Sound; and Government Improvements on the Pacific Coast," by B. W. De Courcy, M. Am. Soc. C. E., Vol. XXV, p. 420.

"Improving the Entrance to a Bar Harbor by a Single Jetty," by T. W. Symons, M. Am. Soc. C. E., Vol. XXXVI, p. 109.

"Description of Coos Bay, Oregon, and the Improvement of Its Entrance by the Government," by William W. Harts, M. Am. Soc. C. E., Vol. XLVI, p. 482.

"Seacoast Harbors in the United States," by C. E. Gillette, M. Am. Soc. C. E., Vol. LIV, Part A, pp. 297, 385.

"Notes on the Bar Harbors at the Entrances to Coos Bay and Umpqua and Siuslaw Rivers, Oregon," by Morton L. Tower, M. Am. Soc. C. E., Vol. LXXI, p. 349.

Mr. Haupt.

"Several years later there was only the original depth of 21 ft. * * *. A new project was adopted extending the jetty 3 miles farther, * * * to be supplemented by dredging out the bar. * * * a permanent depth, of 30 ft. is sought.

"The jetty itself, as now being built, consists of a bed course of small rock as a substitute for the brush mattress, it being found impracticable to use the mattress beyond the shoal depth, * * * and the whole is covered on the seaward slope with very heavy rock ranging in weight from 6 to 16 tons per piece."

These extracts indicate that the usual resources of jetties built out from the shore by the aid of trestles and laid on mattresses, supplemented by dredging, are at least very unsatisfactory, if not impractical, at this locality, because of the activity of the teredo and the waves, so that the method has been modified to conform more closely to that proposed in the recent paper* by H. C. Ripley, M. Am. Soc. C. E.

Notwithstanding the great skill and ingenuity expended in the construction and maintenance of this trestle, more than 6 miles in length, and the excellent system of operation, the physical conditions are such as to retard its advance to such an extent that it seems impossible to overtake the deposits of littoral drift which constitute the bar; for the last official report available at this writing, states the following facts: The estimated cost to secure a depth of 40 ft. was \$3 715 000. This project was revised to raise the jetty to mid-tide level, in 1909, at a cost of \$3 529 300 and to make it 25 ft. wide at that level. In some places it is in 39-ft. depths, thus requiring an enormous increase in the amount of rock. Between September 20th, 1910, and June 30th, 1911, the dredge worked on the bar 62 days, removing 212 080 cu. yd. The project is 85% completed.

"The survey of June shows the channel to have shifted about 2 500 ft. to the northwest, and the depths vary from 25 to 27½ ft., an increase of 1 ft. over last year.

"The life of the trestle is very uncertain. * * * It is believed that the contraction of the entrance by the north jetty may be followed by scouring along the north side of the south jetty (2 miles distant), and that for its maintenance two more groins should be provided, each about 500 ft. long. The total appropriations since 1902 aggregate \$7 901 852.25. The outer 24-ft. contour has advanced some 3 000 ft. since 1902, and the outer depths of 50 ft. in that year had shoaled to 24 ft. by 1909. On the southerly side of the jetty the 24-ft. curve had advanced about 4 000 ft. within 2 years, or about twice the length of the jetty extension in the same time, and large deposits had been made in the throat of the entrance between the jetty and Peacock Spit."

The deductions from these statements are that the extension of the jetty has caused deep erosion at its outer end, requiring enormous

* "How to Build a Stone Jetty on a Sand Bottom in the Open Sea," *Transactions*, Am. Soc. C. E., Vol. LXXV, p. 1040.

quantities of rock to fill voids formerly occupied by sand in place; the rapid advance of the outer slope, accompanied by a shoaling of 26 ft. at the 4-fathom contour; the inability to overtake this advance by the jetty; the shifting of the channel some ½ mile to the northwest by the deposits of drift; the great excess of cost, amounting to more than $1 000 000 per ft. of depth gained; and the proposal to erect the second jetty extending out from Cape Disappointment for a distance of 2 miles, and at this same distance from the present jetty, but neither of them reaching to the crest of the bar by nearly a mile. The result of this must be to aggravate the seaward movement of the bar, while affording little or no protection for dredging. Mr. Haupt.

It is also apparent that the report of 85% completion does not apply to the north jetty, nor to the ultimate extension of both, if this method is to be the main reliance for the improvement of this bar. The purpose of the south jetty, slightly convex to the channel, 4¼ miles in length, was to extend the protecting spit to a point abreast of Cape Disappointment and thus secure the co-operation of that headland, as a second jetty. This has resulted in disappointment, and any extensions of two jetties, it would seem, would give no promise of any different result, because there is no change in the general regimen of the entrance, affecting the relative equilibrium of the flood and ebb currents.

If the life of the piling, due to the teredo, is taken at the maximum limit of 20 months, then the entire structure subject to their attacks, must be rebuilt in that brief time, necessitating constant repairs and heavy expenses for maintenance. Moreover, the statement from actual experience that the dredge was able to work 62 days out of 283, or 22% of the time, covering the winter season, would give a reasonable assurance that it may be quite possible to work on the bar by creating an insular barrier on its outer slope, as a nucleus, sufficient to serve as a breakwater to protect a floating plant. From this a rock jetty could be extended shoreward to connect the deep-water areas on its outer and inner slopes, and to protect the crest from the littoral drift, as well as create a reaction from the impingement of the ample discharge on its concave face which would cut a channel more than 40 ft. deep automatically, as is shown by the existing channel at the base of the Point Ellice headland just within the throat of the entrance, which channel is about 8 miles long and more than 40 ft. deep, having a radius of 5 miles.

So confident is the writer that such a permanent, navigable channel could be readily obtained that, on June 9th, 1902, at the instance of the Senator from Oregon, he filed with the Secretary of War a proposal to guarantee such a channel for the sum of $2 500 000, which proposal was referred to a Special Board of Engineers for consideration and report. On October 14th, the Board gave a hearing to the

Mr. Haupt. officials of the Reaction Jetty Company (organized with a capital of \$1 000 000 to execute the work), at which time a full discussion was submitted to the Board, all the members being present. On November 10th, 1902, the Board requested further information, and submitted twenty-three queries which were categorically answered on the 17th, and after several months of consideration, the report was filed with the Secretary of War, but its contents were not disclosed, and the Reaction Jetty Company was advised that it could ascertain the findings when the annual report of the Chief of Engineers was published. As that report has expunged all reference to the special provisions of the tender then submitted, and as 10 years have elapsed since it was made, and the recommendations of the Board, which were estimated to cost \$2 260 000 for the extensions of the south jetty, at mid-tide level, a distance of 2½ miles, have been executed and for which appropriations approximating \$8 000 000 have been set aside, it would seem appropriate that the terms of that proposal should be reviewed, as a matter of interest to the parties concerned, merely as an index to the early solution of this difficult but important problem of securing an open channel of ample capacity at reasonable cost without bar advance.

The principles involved have been tested on a practical scale at a condemned inlet on the Texas Coast, and notwithstanding the most serious physical as well as "metaphysical" obstacles, a detached jetty actually secured the full predicted depth of 20 ft. before the work was connected with the shores. Then it was seriously impaired, and the channel shoaled to 9 ft., requiring several years before equilibrium was restored. Thus assurance becomes doubly sure.

Briefly, the reaction jetty proposal reviewed the physical condition of the bar and its changes, as shown from official charts, stating that to obtain the 40-ft. channel would require the removal of some 30 000 000 cu. yd. which would be impracticable by dredging in the open sea with no protection works, as it would require some 30 years, if there were no littoral drift, to maintain the supply, and would cost more than \$6 000 000.

To control this drift and create an automatic channel, the company designed a permanent structure to create a zone of local activity across the bar, to arrest the drift on its convex face, being on the windward or "weather" side of the proposed crossing, and, at the same time, to cause a continuous reaction, with erosion and deepening on its concave face, by utilizing the forces and agencies of impact, head, reaction, concentration, gravity, and continuous deflection, whereby a sufficient amount of the potential energy of the affluent water is developed locally to produce deep scour and lateral transportation.

The old jetty, completed in 1895, which created the temporary

30-ft. depth, had caused the bar to shoal up at the rate of 3 500 000 cu. yd. per annum, and pushed it seaward about 1 mile, reducing depths of 60 ft. to 30 ft., and less. Mr. Haupt.

To avoid this advance, the company proposed to make its work discontinuous, leaving a gap of about 2 miles between the outer end of the existing jetty and the inner end of the breakwater. The company also proposed to reverse the direction of the curvature of the breakwater, so as to place its cutting face toward the ebb currents and to receive them at first tangentially and, by gradual but continuous deflections, cause a constantly increasing pressure on the most vital part of the ebb for a distance of about 2 miles across the bar. Thus, too, the ingress of the flood tide would not be impaired, and the conditions of equilibrium would be changed in favor of the ebb stream.

Numerous other advantages, not incidental to straight jetties, were pointed out, time and expense were to be saved, and no risk was to be incurred by the Government, which was to be fully safeguarded by bonds, and pay only for work in place and depths secured.

Plans and cross-sections of the existing and proposed channels were submitted, with the form and dimensions of the proposed structure, closing with these words:

"As the proposed plan is novel and the location one of exceptional difficulty, this Company desires, if permitted, to accept all the responsibility for the success or failure of its plan, and will furnish such security as the Secretary of War may require to protect the United States from loss by reason of its failure. It will also satisfy the Secretary of its ability to carry out the contract to execute these plans as rapidly as the physical conditions will permit * * *. We believe that this proposition will be found to be 'the most economical and advantageous to the Government,'* since it is based upon the utilization of a part of the enormous energy now going to waste over the bar."

After summing up the special advantage to be secured by the acceptance of the proposal, the Company pledged its good faith for its execution in the following words:

"If this Board, therefore, can see its way to recommend to the War Department the acceptance of this proposition for the relief of the Columbia River Basin, it is believed that it will perform a public service of great value to the country, and the Company will accept the trust and the responsibility of its execution in good faith, and will release the Government from any or all claims for royalties incidental to the use of its patents at this place."

The sequel to this, and similar tenders with like guaranties, indicated so plainly that the policy of past administrations was not to

* Terms of the law authorizing the Secretary to make such contracts.

Mr. Haupt. encourage the letting of contracts to extrinsic parties, however guaranteed, that the Reaction Jetty Company was dissolved.

The changes on the bar during the past 10 years have served to confirm the predictions made to the Board of 1902 as to the probable effects of the Government plan, and the elaborate report submitted at that time only alluded to the offer of the Company in a single paragraph as follows:*

"The Board knows of no plan for the improvement of this entrance that has not contemplated one or two jetties extending continuously seaward from the points of the entrance that are fixed in position naturally or artificially. The turbulence of the bar is such that operations from floating plant have never been seriously considered as practicable, and any work must be executed from a structure built out from the shore. Even the structure proposed by the Reaction Jetty Company, while nominally a detached breakwater, is in effect an extension of the old jetty, * * *."

This was the only reference, and the extension of the south jetty, as since constructed, was recommended, notwithstanding the following statements in the report:

"The Board cannot expect that an advance of the bar at this point will not follow the construction of the jetties and the removal of the large quantity of sand necessary to secure a 40-ft. channel. Such advances have invariably been found in all jetty harbors."

But it is expected that:

"The waves and strong littoral currents have at this point their maximum effect, in retarding and counteracting the bar advance, and, unless the history of the past 60 years is misleading, that advance will be speedily checked and probably reversed with a return of the outer bar slope toward its present location."

These hopes do not appear to have been justified by the results in the later reports, and the question arises as to whether or not there may ever be a better solution than the two-jetty plan, which, it is conceded, "invariably" advances the bars in "all" cases. The answer may be suggested by the experience at Aransas Pass during the operation of the partial reaction breakwater, which not only prevented bar advance but caused a recession of the outer contours until the channel was cut entirely through, without injury to the structure, by a feeble diurnal tide. There are many other elements in this report of the Board of 1902-03 which are worthy of consideration, but space and time prevent the writer from mentioning them.

It is hoped that these suggestions may open the door to a broader consideration of the policies and possibilities of this nation for removing physical obstacles to international trade, and be of greater economic advantage to all people.

* Report, Chief of Engineers, U. S. A., 1902, p. 2305.

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PAPERS AND DISCUSSIONS

TUFA CEMENT, AS MANUFACTURED AND USED ON THE LOS ANGELES AQUEDUCT.
Discussion.*

By Messrs. J. M. O'Hara, L. J. Le Conte, and Ralph J. Reed.

J. M. O'Hara, Assoc. M. Am. Soc. C. E. (by letter).—According to Professor Eakle, of the University of California, the rock ground with the cement used on the Los Angeles Aqueduct, is a rhyolite-tuff and a trachyte-tuff, which, when finely ground, will possess the same characteristics as finely ground clay. Mr. O'Hara.

The cement produced by the method used at Monolith and Haiwee, Cal., is similar to cement adulterated with clay. Rhyolite-tuff and trachyte-tuff are not to be confounded with the volcanic rocks known as puzzuolana and trass, which have been used for the manufacture of cement. Puzzuolana and trass are the hardened products of volcanic action in their original state, in which respect they are similar to blast-furnace slag. On the other hand, the volcanic tuff of the nature found at Monolith and Haiwee, is not comparable to blast-furnace slag, being more of the nature of altered volcanic rock.

In Professional Paper No. 28, issued by the United States Government, through the Corps of Engineers, it is stated:

"Puzzolan cement never becomes extremely hard like Portland, but Puzzolan mortars and concretes are tougher or less brittle than Portland. * * * It is unfit for use when subjected to mechanical wear, attrition, or blows. It should never be used where it may be exposed for long periods to dry air, even after it is well set. It will turn white and disintegrate, due to the oxidation of its sulphides at the surface under such exposure."

A series of tests made at the Ohio State University, under the direction of Professor C. E. Sherman, seems to prove that clay, up

* This discussion (of the paper by J. B. Lippincott, M. Am. Soc. C. E., published in October, 1912, *Proceedings*, and presented at the meeting of December 4th, 1912), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. O'Hara. to 15% of the weight of the sand, adds strength to the mortar. One plausible explanation for this increased strength is that the addition of smaller particles of material aids in filling the voids in the sand. One detrimental result was definitely established, namely, that mortars made from sand carrying clay could not be placed under water safely, because the clay softened and warped under its influence.

Mr. Lippincott states:

"As tufa cements are high in silica, and as the silicates of lime are the more enduring but slower portion in the cements, this growing strength in tufa cement is quite rational. Straight cements which are slow in hardening show the greatest ultimate strengths, and a high 7-day test is regarded with suspicion."

Tufa and Portland cement, as used on the Los Angeles Aqueduct, is a mechanical mixture, the two materials being blended in equal parts by volume. Under these conditions, no silicates of lime are found, and the gradual increase in strength is not due to the same cause as the slow hardening of a high silica Portland cement.

In setting under water, especially sea water, the lime set free from cement by the action of the water combines with soluble silicates, and maintains the strength of the mass. The percentage of lime set free is small, however, and in the Aqueduct product would not combine with much of the silica. When the concrete sets in air, little or no combination with the tufa will take place, and it is only present as an adulterant.

In the manufacture of Portland cement to-day, the clinker is ground much finer than was the custom some years ago, and the lime factor is higher. Consequently, the calcium silicates hydrate more readily, and ultimate tensile strength is reached at an earlier period. Besides, the flash strength, due to the calcium aluminates or gypsum, should not be mistaken for the final strength, due to the calcium silicates.

A high 7-day strength is not of itself an indication of poor quality in a cement, all other things being normal; it may be an indication of early ultimate strength.

The writer is of the opinion that the trend of the paper, to show that the tufa cement, as made at the Los Angeles Aqueduct, is as good as a Portland cement, is dangerous and without precedent.

Mr. Le Conte. L. J. Le Conte, M. Am. Soc. C. E. (by letter).—The natural property possessed by tufa of combining with the free lime in all Portland cements, is certainly a very great discovery, the full importance of which cannot be over-estimated. This interesting feature is further emphasized by the announcement of Dr. Michaelis that his tests, extending over five years, show conclusively that the addition of tufa to lean Portland cement mortar is valuable in sea-water, this latter being especially important to harbor engineers. In view of the general

experience that Portland cement concrete slowly disintegrates in sea-water, Italian harbor engineers usually recommend a mixture of puzzuolana and Portland cement for all concretes exposed to its ravages. In many cases where unit strength and quick setting are uncalled for, they use a mixture of ordinary fat lime and puzzuolana. This shows a tendency to return to the old Roman practice, long before the time of Vitruvius, 14 B. C., and recalls the old adage: "Verily, the footprints of the old Roman engineers are eternal." Mr. Le Conte.

The writer's attention was first called to the fact of the falling off in strength of Portland cement briquettes after 1 year, while on the fortifications at and near Fort Point, San Francisco, Cal., in 1895 to 1897. At that time some 350 000 bbl. of European Portland cements had been used. The records showed a general falling off in strength after 1 year, a few samples extending to 10 years. The same feature was noticed in the reports of the Metropolitan Water Board, Boston, Mass., and seems to be the experience everywhere.

The author's experiments, indicating that sand briquettes containing 50% of tufa cement showed marked superiority in ultimate strength over those with straight Portland cement after 6 months, are certainly most encouraging.

Years ago, the writer decided that the well-known "Dyckerhoff" brand of Portland cement was the best in the market, but its use was prohibited by its high cost due to transportation expenses from the works to the seaport. In private works under his charge, he overcame this difficulty with a 1:5:10 concrete, which gave highly satisfactory results. It would be exceedingly interesting to know how tufa cement in the same proportions would behave.

There seems to be a fad for demanding "sharp coarse sand" in specifications for concrete. This fad is based on the general craze for short-time high tests, and nothing else. Engineers often go to great expense to get good coarse sand to make a high-test record, when, in point of fact, after 3 months, briquettes made with fine beach sand show just as great strength as those made with coarse sand, if not greater; that is, after the expiration of 3 months, all the advantages of using coarse sands have entirely disappeared. The cost of manufacture given by the author is most encouraging, and puts new life into the industry.

The lining of irrigation canals in leaky ground is certainly a most serious question, and all sorts of expedients are being tried to overcome the practical difficulties. The proposed lining with lean concrete seems to be the best solution of the problem. The experiments with Kieselguber are very instructive, and broaden the entire field of investigation.

RALPH J. REED, JUN. AM. SOC. C. E. (by letter).—It is obvious that, on account of finer grinding and consequently better mechanical Mr. Reed.

Mr. Reed. combinations with other aggregates, tufa cement must make a more dense and impervious concrete. It would have been interesting to have presented some experimental data showing comparisons between the permeability of straight Portland cement concretes and those made with tufa Portland cement. Doubtless experiments along these lines have been carried on in the Los Angeles aqueduct laboratories.

The writer wishes to bear witness to the excellent appearance in general of the concrete work on the aqueduct. A large portion of this work has been constructed with tufa cement concrete. By far the largest portion of the work has been completed under conditions which those at all familiar with Western deserts will recognize as far from ideal for the most successful concrete work. It has apparently been difficult at many points to obtain first-class aggregates. Water for sprinkling the finished work has been hard to supply, and, in fact, the many difficulties encountered are appreciated only by those connected most closely with the work. The writer has been especially impressed with the freedom from cracks shown by the finished tufa cement concrete work in tunnels, open and covered conduits, and especially in the large reinforced concrete siphons referred to by Mr. Lippincott.

That the tufa cement must be handled with greater care during drying is apparent from the fact that in the vicinity of open manholes along the covered conduit in the Mohave Desert, where the concrete during the later stages of drying has been exposed to the sun, cracks are frequent. A little farther back, where the atmosphere has been humid on account of water left standing in the conduit, very few cracks are noticeable. In the tunnels, where the atmospheric conditions and especially humidity have been most completely in control, the tufa concrete lining is dense, hard, and tough, and rarely shows any cracks.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

THE ECONOMIC ASPECT OF SEEPAGE AND OTHER LOSSES IN IRRIGATION SYSTEMS.

Discussion.*

By Messrs. L. J. Le Conte and W. C. Hammatt.

L. J. Le Conte, M. Am. Soc. C. E. (by letter).—The curable and incurable losses of water in an irrigation system are certainly worthy of the most careful study and judicious consideration. The loss due to leakage from the reservoir bed is usually regarded as incurable, nevertheless, the site ought to be most carefully examined, geologically, before it is finally accepted. Mr. Le Conte.

Experience everywhere shows that when a large storage reservoir is built, the knowing ones always buy the water rights on the adjoining stream on the lower side. They know from experience that the leakage from the reservoir will swell the run-off of the adjoining stream. Where the stratification dips naturally from the reservoir site toward the stream in the adjoining water-shed, the leakage may be very serious. A notable case is that of the River Glyde, in Ireland, where the rainfall and run-off were being carefully observed by able engineers. Observations for 3 months in the rainy season showed a rainfall of 5.89 in. and a run-off of 9.35 in. This result gave rise to a great deal of merriment, at the time, but subsequent observations proved that the flood-waters from the higher adjoining water-shed found underground passages and escaped into the Glyde water-shed, thus swelling the observed run-off to abnormal dimensions. This was discovered by putting coloring matter in the upper stream and noting its appearance in the waters of the Glyde at a lower level, where the gaugings were being made. This simple experiment explained the whole mystery.

* This discussion (of the paper by E. G. Hopson, M. Am. Soc. C. E., published in October, 1912, *Proceedings*, and presented at the meeting of December 4th, 1912), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Le Conte.

This only goes to show how fallacious it is to assume that the run-off records show all the water that naturally comes from the direct water-shed. In point of fact, a large percentage of it may come from an indirect water-shed. A notable case is that of the Danube River; observations at the Iron Gate show that a large percentage of the summer flow goes through subterranean channels, and swells the summer flow of another stream in an adjoining water-shed.

It is quite common in every-day practice to pick out a dam site where a dike cuts across the valley, the presumption being that all danger of underground flowage is thereby avoided. This is true, but, at the same time, it makes a fine face for leakage to run along over and into the adjoining water-shed.

Mr. Hopson's estimate of a loss of 55% seems to be extravagant. In designing new works, it is generally customary to allow for a loss of one-third of the total, two-thirds of the water being delivered at its final destination. Of course, when the canals are first opened to service, the loss is very great (fully 55 to 60%); but it gradually grows less and less as the channels silt up. Shortly after the works are opened, the maintenance force begins the work of puddling the canal beds. This, of course, is usually done at times when the water is least needed for irrigation, and duties are not pressing. This puddling is kept up each year, until it is completed, when the total loss from seepage and evaporation is generally reduced from 60 to $33\frac{1}{3}$%, but as low as 25% in some cases. In a majority of cases, the quantity of water saved by the canals being lined or unlined, but puddled, as usual, is not to exceed 25% at best, and may be much less; hence, lining is certainly questionable. Of course, in all cases where water is highly valuable, the scheme of lining the canals is entirely feasible and desirable; but, in pioneer countries which are being developed, the first cost is practically prohibitive, and, as a rule, that work is left to future generations.

The important point made by the author, however, that the cost of an unlined canal system and a lined canal system will be practically the same, certainly calls for full investigation. At present the writer is unable to see it in the light presented. The author also calls attention to the great saving in drainage troubles, brought about by a general and complete system of lined canals, laterals, and distributing ditches. There seems to be little doubt that the drainage troubles would be greatly mitigated; at present, they constitute the most distressing feature of irrigation works, the unsanitary conditions created and developed by the best types being notorious.

In most cases surface and sub-drainage take care of themselves. The natural drainage channels should be used for that purpose exclusively. They should be cleaned out and some little money should be spent in deepening, straightening, and correcting any natural defects.

Mr. Le Conte.

The lamentable effect of excessive irrigation, on the one hand, and defective drainage, on the other, are beyond ordinary comprehension. When all the facts of any case are properly grasped, the drainage problem naturally develops into a veritable sink-hole for the waste of public money; hence the necessity for the exercise of the highest grade of good judgment.

In India the British Government was compelled to pass the most drastic laws to control irrigation.* On sanitary grounds, no water is allowed to be issued for autumn crops nearer than:

5	miles	from	a	military post.						
1	mile	"	"	native	town	of	more	than	10 000	inhabitants.
½	"	"	"	"	"	"	"	"	5 000	"
¼	"	"	"	"	"	"	"	"	1 000	"

200 yd. from small villages.

Mr. Hammatt.

W. C. Hammatt, M. Am. Soc. C. E. (by letter).—The question brought out in this excellent paper is of great interest to irrigation engineers. It is really, however, a business proposition, as to how much expense is justifiable for the purpose of preventing, or diminishing, the losses and damage due to seepage from canals and ditches. Into this determination enter so many factors, that no rule or formula, however complex, can be made to cover the subject.

The quantity of seepage from canals has been the subject of many investigations, and is dependent on many conditions, which conditions will affect its cure. The writer knows of cases where, at certain seasons of the year, the seepage into a reservoir from its water-shed more than offsets the evaporation therefrom, so that the reservoir remains practically stationary under a 120° sun.

In both reservoirs and canals, the seepage loss is dependent on the depth of water in the canal, the breadth of the wetted perimeter, the soil through which the canal is excavated, the kind of subsoil and the distance thereto, the height of the ground-water, the slope of the country, the growth of weeds in the canal, the character of the vegetation outside of the canal, and many other factors. All water which seeps from the canal can be accounted for and traced to its destination. In some cases it has a flow through the soil, in a definite direction and at a determinable speed, toward some open watercourse or underground reservoir. The character of the soil through which the canal is cut may make a variation in the rate of seepage of from 0.3 to 1.6 vertical ft. per day—the limits of measurements made by the writer. Aquatic grasses and weeds require water for their growth, aid seep-

* See discussion by Surgeon-General H. W. Bellew on paper "The Evils of Canal Irrigation in India, and Their Prevention," by T. H. Thornton, *Journal*, Society of Arts, Vol. XXXVI, p. 521 (Mar. 23d, 1888); and "The Injurious Effects of Canal Irrigation on the Health of the Population of the Punjab, and Their Remedy," by Surgeon-General H. W. Bellew, *Journal*, Society of Arts, Vol. XXXVI, p. 640 (May 11th, 1888).

Mr. Hammatt. age by loosening the soil and facilitating the passage of water to the subsoil, and increase evaporation through their leaves. Measurements under the supervision of the writer have shown this to increase the natural seepage as much as 40 per cent. A certain quantity of seepage goes to supply the wants of near-by vegetation, thus creating a flow and a gradient in that direction.

The seriousness of the loss by seepage depends on various causes, among which are the following: The scarcity of water and consequent loss in crop value, due to lack of the quantity lost by seepage; the character of the soil, and the consequent tendency toward the rise of the ground-water; and the character of crops, and the effect on them of a high or low water-table.

In Southern California, where water is scarce and the crops consist mainly of citrus fruits, the value of the water for crop propagation will pay for an immense expense for seepage prevention. On the other hand, in Central California, where water is plentiful and the crops are grain, cereals, and alfalfa, and $5 per acre per year is a prohibitive price for water, very little expense for the prevention of losses is justifiable. It is seldom that the seepage from a canal is sufficient to water-log the soil. At the maximum rate the writer has seen, namely, 1.6 vertical ft. in 24 hours, the seepage from a canal carrying 100 sec-ft. would be about sufficient per mile to raise the ground-water 1 ft. in 900 acres. As the 100 sec-ft. would irrigate about 16 000 acres, and as only about 10% of this would go toward crop propagation, the remainder staying in the soil, the small proportion of ground-water due to canal seepage is apparent. Consider also that the farmer's tendency is to over-irrigate, especially where water is sold at a flat price per acre without regard to the quantity used, and we have the reason for the drowning of so many good farming areas.

B. A. Etcheverry, Assoc. M. Am. Soc. C. E., has made a great study of various linings for canals and ditches.* The data which he has compiled show a maximum cost of lining of about 6 cents per sq. ft., and this is for thin mortar lining which is only adaptable to canals in solid ground capable of resisting the pressure, and with no possibility of settlement. In most of the canal systems of Central California, the cost of the entire system, exclusive of the preparation of the land itself, has fallen to between $10 and $25 per acre of irrigated land. The cost of lining only the main canals and branches would raise the cost of the systems from 25 to 50%, which would only be justified by a continued scarcity of water.

The writer has only discussed the lining of canals for the prevention of seepage losses. Where the point of diversion is so much

* These data may be found in *Bulletin No. 188*, Agricultural Experiment Station, University of California, and in *Bulletin No. 44*, Department of Agriculture, Victoria, B. C.

Mr. Hammatt.

above the point of use of the water that a steep grade and a swift flow is obtainable, the lining of the canal—by reducing the coefficient of roughness and by increasing the allowable velocity—will reduce the section sufficiently to repay its cost, partly, if not wholly. Another element is the growth of weeds, which would be eliminated by a concrete lining.

To summarize: The question as to the lining of canal systems should be decided by balancing, against the cost of such lining, the value of the water lost, the damage done by that water, the cost of excavation and of structures saved, and the elimination of the cost of canal cleaning.

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To summarize: The question as to the lining of canal systems should be decided by balancing, against the cost of such lining, the value of the water lost, the damage done by that water, the cost of excavation and of structures saved, and the elimination of the cost of canal cleaning.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

SPECIFICATIONS FOR METAL RAILROAD BRIDGES MOVABLE IN A VERTICAL PLANE.

Discussion.*

By Augustus Smith, M. Am. Soc. C. E.

Augustus Smith, M. Am. Soc. C. E. (by letter).—The writer desires to add his acknowledgment of the value of the specifications for movable bridges,† by C. C. Schneider, Past-President, Am. Soc. C. E., and of the discussions contributed by other members of the Society on that subject. There was great need for definite specifications, and Mr. Leffler's valuable contribution is another important gift to the Profession. Mr. Smith.

The writer is glad to discuss a few of the details defined so unequivocally by the author, taking them up in order, as follows:

Paragraph 55.—The author apparently favors the use of tapered keys, but obviously could not attempt, in the text of a specification, to give any justification for this preference. Without rehearsing the arguments against tapered keys,‡ the writer would call attention to the fact that they cost more than straight keys and require considerable room on the shaft for each wheel fitted, so that it is not practicable to place a wheel close to a bearing, and, when such keys are used, it is not practicable to place two or more wheels very close together. In some cases tapered keys are necessary. The writer's practice is to use straight keys where possible, and tapered keys where straight ones are not practicable.

Paragraph 92.—Worm gearing is used frequently for transmitting power, where it is desirable to hold the load. If the angle of the

* This discussion (of the paper by B. R. Leffler, M. Am. Soc. C. E., published in October, 1912, *Proceedings*, but not presented at any meeting), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† *Transactions*, Am. Soc. C. E., Vol. LX, p. 258.

‡ Summarized in Kent's "Mechanical Engineer's Pocketbook."

Mr. Smith. thread for the worm were 20° or more, the wheel would be able to turn the worm backward. There are not many data on the limiting angle of thread which will give the maximum efficiency of a worm drive without being steep enough to permit the wheel to drive the worm. The writer has found the angle to be approximately 6°, but much depends on the nature of the thrust bearing holding the worm, and the angle would be less if first-class ball bearings were used.

Paragraph 136.—The author allows for the bending stress in the individual wires of rope bent around a sheave, and points out that this is a function of the diameter of the individual wires and not directly of the diameter of the rope, as it is sometimes apparently thought to be. The same thing has been worked out by the engineers of the Trenton Iron Company.* Attention is called to the fact that the minimum diameter of sheaves and drums given in the lists of all manufacturers of wire ropes has been apparently agreed on, like the price list, but it is not altogether consistent with the foregoing theory.

The writer confesses to some obtuseness in his power of conception of the action of cumulative stresses. If the bending stress in the wires is really added to the tensile stress produced by the load on the rope, an overloaded rope should always break on a sheave or drum, and not in a straight part.

Paragraph 140.—The author's formula for determining the strength of gear teeth follows the general form advocated by Mr. Lewis, namely, by making the allowable pressure proportionate to the pitch and face multiplied together, instead of the earlier formula given by Tredgold, in which the pitch was squared. The author's formula makes due allowance for the number of teeth and the velocity, and is intended to apply to steel gearing, because, according to Paragraph 38, only cast or forged steel should be used for this purpose. He should add a formula fixing the face in terms of the tooth pressure, to provide for wear. Cast iron works with less friction, and usually with less tendency to cut, than cast steel or forged steel when running together, and the writer is doubtful about allowing much greater tooth pressure per inch of face for steel than for first-class close-grained cast iron. It will be observed that the permissible pressure on the contact surfaces between the teeth of gears given in Paragraph 140 is many times greater than that allowed in sliding contact by Paragraph 150. Unwin mentions the old rule that the pressure between teeth should not exceed 400 lb. per in. of face in order to obtain good wearing results. The late James Christie,† M. Am. Soc. C. E., in his discussion of Mr. Schneider's paper, gives a formula naming the working pressure of the tooth in pounds per inch of face, in terms of the pitch multiplied by a constant dependent on the nature of

* The results are given in the handbooks of that company.

† *Transactions*, Am. Soc. C. E., Vol. LX, p. 318.

the material, and divided by the square root of the velocity. Mr. Christie's formula, it will be observed, gives higher values for steel than for cast iron, in the ratio of 10 to 6. Mr. Smith.

Paragraph 157.—The writer would suggest adding to this paragraph a sentence to the effect that when brakes which act through the transmission machinery are used, all parts of that machinery so affected shall be designed to withstand the maximum force set up by the application of the brakes, if it should be more than that of the driving motor.

Paragraph 168.—The author limits the maximum piston speed of internal combustion motors to 350 ft. per min. A comparison of the various speeds given by manufacturers of so-called "heavy-duty" motors would indicate an allowable piston speed of 500 ft. per min., and even higher. The writer's experience with a gasoline motor indicates perfectly satisfactory results at 500 ft. per min. One objection to putting the piston speed too low in motors of this type is the inevitable leaks past the piston rings, and at low speeds there is a marked loss in compression around the valves, so that an internal combustion engine does not develop the proportional power at a low speed that it would at a higher speed.

The writer would also recommend adding a provision to this paragraph limiting the ignition, if electric, to the jump-spark method, and would also advocate specifying the use of a jump-spark apparatus in which the secondary coil is made up on each spark plug as part of it, so that a low-tension current of 6 or 10 volts at the outside is all that would have to be taken care of in the wiring. He would recommend a clause prohibiting the use of any internal combustion motor in which cooling water was passed through a packing or joint designed to withstand the gases of combustion. This provision, in the case of engines in which the cylinder head is made separable from the body of the cylinder, would require the cooling water to be by-passed outside from the cylinder to the head, or brought to and from the head and cylinder separately. Preference should be given to an air-cooled motor of proven reliability.

Paragraph 177.—In this paragraph the author names the spare parts to be furnished with an electric motor. The writer thinks that certain spare parts, such as igniters and crank-pin brasses, would be advisable in the case of internal combustion motors.

Paragraph 180.—The author specifies controllers of the reversible drum type, and while these give very good results, the writer has found that controllers of the disk type, such as are made by the Electric Controller and Manufacturing Company, give equally satisfactory results in hoisting and crane service, so that he would not advise limiting controllers to the drum type.

In the discussion of Mr. Schneider's paper a number of leading

Mr. Smith. designs of bascule bridges were brought out. Though it has no direct bearing, as part of this discussion, it might be of interest at this time to add to the various bridges brought out by Mr. Schneider's paper, a design which the writer prepared for the Stone Bridge, at Tiverton, for the State of Rhode Island, based on the old Delille type of fortification draw-bridge.

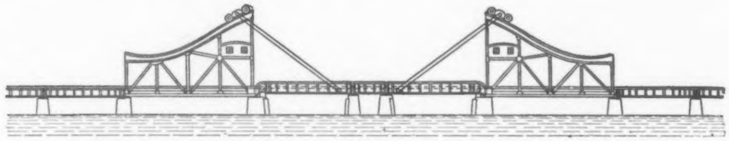

FIG. 3.

The mechanical principles involved in this bridge are obvious from Fig. 3. The advantage of the stiff connecting link between the moving bascule and the counterweight is that the bridge can be balanced at all points and forced down against a wind pressure. Another advantage is that the spans on which the counterweight moves can be used frequently as through truss spans, like approach spans, on each side of the draw, with some economy in metal. A third feature is that all the moving parts are high above the water, and cannot be affected by salt spray or ice. In common with many others of the bascule type, a low-level bridge is possible, and no expensive foundations for counterweight or the like are necessary. Another advantage—and it seems to the writer a very important one—is that this bridge, especially where a double-leaf bascule is used, has a more pleasing architectural effect than is commonly the case with bascule bridges.

In the Tiverton Bridge, which was designed for highway and trolley car service, no central pier was used, but the connecting rods were made of very ample cross-section so as to reduce the deflection of the bridge at the center to a negligible amount for the purpose. For railway service, a center pier of some sort would doubtless be desirable; such a pier would be built in the middle of a two-leaf span with less obstruction to traffic than would be the case if a swinging draw were used.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

THEORY OF REINFORCED CONCRETE JOISTS.

Discussion.*

By Ernest McCullough, M. Am. Soc. C. E.

Ernest McCullough, M. Am. Soc. C. E. (by letter).—In some recent correspondence with Sanford E. Thompson, M. Am. Soc. C. E., the writer's attention was called to a too common error on the part of designers in reinforced concrete, wherein the sum of the middle bending coefficient and the coefficient at one support is always assumed to be equal to $\frac{1}{8}wl^2$. The moment of inertia of the section is not considered as being affected by the quantity of steel, the result being that, if a middle coefficient $= \frac{1}{10}$ is assumed, the coefficient over supports is taken as $\frac{1}{40}$. The writer checks a great many plans for contractors and owners, these plans being generally furnished free of cost by steel salesmen. The almost uniform practice is to use the clear span, face to face of T-beams, for floor slabs, and the clear span, face to face of columns, for beams and girders, the foregoing coefficients being used for steel distribution. The writer has never been able to make the designers do more than use a coefficient of $\frac{1}{12}$ in the middle and $\frac{1}{24}$ over supports. He has sometimes failed to secure a concession of even that much, as the designers can refer owners to a large number of buildings, erected in every State in the Union, designed with the foregoing dis- Mr. McCullough.

* This discussion (of the paper by John L. Hall, M. Am. Soc. C. E., published in October, 1912, *Proceedings*, but not presented at any meeting), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. McCullough.

tribution of bending coefficients. When possible, he compels the use of $\frac{1}{12}$ over supports and $\frac{1}{18}$ in the middle, whereas, the building ordinances of Chicago and most cities call for a total of $\frac{1}{6}\,wl^2$, with a coefficient of $\frac{1}{12}\,wl^2$ in the middle, of course, then obtaining equal steel areas in the middle and over each support.

Two weeks ago a designer gave as authority for his method, Turneaure and Maurer's "Principles of Reinforced Concrete Construction," second edition, page 307, where the following occurs:

"* * * but the use of $\frac{1}{10}$ for the general coefficient will provide ample strength in all ordinary cases. This would require an actual resisting moment at each support of only about $\frac{1}{40}\,pl^2$ in order that the center moment be reduced to $\frac{1}{10}\,pl^2$."

His attention was called to a line on the same page where the following appeared:

"The foregoing calculations assume uniform moment of inertia and therefore that about the same amount of steel is used for negative as for positive moments. The effect of variation in moment of inertia is discussed in Art. 166."

In the preceding paragraph, on the same page, the authors propose certain coefficients, saying plainly that they are for both positive and negative moments. To the writer, it seems impossible to make any mistake in the matter, if it is carefully studied, even with no other authority than that above quoted.

Turning to Art. 166 in the above-mentioned book, it is found to deal with columns, the reference being a misprint, Art. 165 being meant. This section is a valuable discussion of the effect of varying moments of inertia, being based, for the most part, on a discussion by P. E. Stevens, Assoc. M. Am. Soc. C. E.* A table is given, based on the moments of inertia varying with the quantities of steel, showing that if the assumption be made that the middle coefficient $= \frac{1}{8}\,wl^2$ and about one-fifth of the quantity of steel in the middle in the bottom be placed in the top over supports, the stress at the center will be 55% of the working stress and at the end will be $4.1 \times 0.55 = 2.25$ times the working stress.

In the last edition of "Concrete, Plain and Reinforced," by Taylor and Thompson, the following is found on page 439:

* *Transactions*, Am. Soc. C. E., Vol. LX, 1908, p. 496.

"In applying this to the various cases, the assumption is made that the moment of inertia of the beam is constant throughout its length. While this is not strictly true, extensive studies of various cases in reinforced concrete show that a large change in the moment of inertia makes a very small change in the bending moment, so that the relations are substantially correct until a member enters a much larger member." Mr. McCullough.

Mr. Thompson, in a letter to the writer, says:

"Many designers have the very erroneous idea that, because they design a beam for $\frac{wl^2}{8}$, the bending moment is reduced to zero over the supports. This, of course, is not the case, because designing the beam for $\frac{wl^2}{8}$ is an entirely different matter from changing the bending moment to $\frac{wl^2}{8}$. The bending moment still remains in the neighborhood of $\frac{wl^2}{24}$, if the beam is fully continuous; so that the bending moment at the support for uniform load is still a little less than $\frac{wl^2}{12}$."

In his paper, Mr. Hall has placed the matter in the proper light, but has neglected to call attention specially to the fact that all concrete designs, wherein continuity of construction is considered, should be based on the end moments rather than the center span moments.

A few days ago, in conversation with three structural engineers, members of this Society, the writer discovered, to his surprise, that they believed the sum of the center and one end coefficient $= \frac{1}{8}$; and if the assumption is made for a large bending moment in the center, that only enough steel is required over supports to care for the remainder of the coefficient, it being their practice to use arbitrarily over supports about one-fifth of the quantity of steel used in the middle, in the event that they use $\frac{wl^2}{8}$ in the center. This small quantity of steel they claim is to take care of possible cracks, mainly caused by temperature, and possibly by a bending down of the flanges of T-beams, or unlooked for variations in loading. As such men hold to these views, it is proper to have a full discussion of this phase of the subject, and that some present methods of designing be discouraged. Five or six years ago such views were common, and it is surprising that by this time so many men of standing in concrete work still neglect consideration of the fact that "designing the beam for $\frac{wl^2}{8}$ is an entirely different matter from changing the bending moment to $\frac{wl^2}{8}$," as stated by Mr. Thompson.

[illegible]

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PAPERS AND DISCUSSIONS CURRENT IN PROCEEDINGS